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PROCEEDINGS VOL. 54

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Coming next month in the JOURNAL

New design methods are presented by Charles S. Whitney in "ULTIMATE SHEAR STRENGTH OF REINFORCED CONCRETE MEMBERS WITHOUT SHEAR REINFORCEMENT." Radically different method for the ultimate strength design for shear of flat slabs, column footings, beams, and frames is supported by results of tests covering a wide range of proportions, and concrete and steel strengths.

"LIGHTWEIGHT-AGGREGATE CONCRETE FOR STRUCTURAL USE" by J. J. Shideler describes the tests employed and the results obtained in an extensive investigation of the properties of lightweight-aggregate concrete of structural quality.

George L. Dahl tells of the "CONSTRUCTION OF THE DALLAS MEMORIAL AUDITORIUM." The all-purpose building that can be used for conventions, operas, circuses, sports events, and various exhibitions is a circular structure with a thin shell concrete dome roof supported on columns at the perimeter of the building.

"STUDY OF ULTIMATE TORSIONAL PROPERTIES OF RECTANGULAR REINFORCED CONCRETE BEAMS" by G. C. Ernst endeavors to determine the quantity of transverse steel required to develop the yield point in longitudinal bars placed in the corners of rectangular beams subjected to pure torsion. Results of 18 tests are reported.

G. W. Washa and J. C. Saeman are co-authors of the paper, "VARIATION OF MORTAR AND CONCRETE PROPERTIES WITH TEMPERATURE," which represents a detailed study of the effects of temperature upon strength, stiffness, and toughness of mortars and concretes. Test temperatures varied between −70 and 450 F.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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Title No. 54-11

Introducing a series of reports on the ACI Code

Review of Changes in the ACI Building Code Requirements for Reinforced Concrete*

By FRANK KEREKES†

SYNOPSIS

The philosophy of building codes is treated briefly. Evolution of the ACI Building Code and its contents are reviewed, and then related to teaching and to practice. The changes incorporated in the 1956 Code are summarized.

THE PHILOSOPHICAL BASIS OF BUILDING CODE PREPARATION AND USE

Fulfillment of an idea

No one may ever care to know how many hours of toil and critical thinking were contributed freely and zealously by hundreds of theoretical and practical engineers in creating an internationally recognized standard of reinforced concrete building practice in the 73 pages of "Building Code Requirements for Reinforced Concrete (ACI 318-56)." No one will ever determine the thousands of engineers, contractors, and architects who were, and are, guided by the ACI Building Code in the design of billions of dollars worth of substantial and noteworthy reinforced concrete buildings and related structures.

Without the cumulative and crystallized experiences furnished by a host of competent designers, builders, and materials experts, the inexperienced beginner, as well as the seasoned practitioner, would be deprived of the benefits of the vast amount of proven and authoritative counsel placed at his disposal by ACI 318-56, which represents an evolutionary development over a period of 50 years by members of Committee 318 of the American Concrete Institute.

Evolution of code content

The present ACI Code was developed by the intensive effort of 44 committee members who worked continuously and effectively through correspondence and meetings over a period of 5 years since the 1951 issue of the Code. Each change, addition, and/or deletion was first thoroughly discussed in subcommittee, and then deliberated upon by the committee as a whole until final agreements—so essential to the production of a generally accepted document of engineering prestige—were officially achieved.

*Adapted from a report presented at the Fourth Illinois Structural Engineering Conference, University of Illinois, Dec. 1956. Received by the Institute Jan. 7, 1957. Title No. 54-11 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 3, Sept. 1957, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Dec. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, chairman of committee which developed "Building Code Requirements for Reinforced Concrete (ACI 318-56)," Dean of the Faculty, Michigan College of Mining and Technology, Houghton, Mich.

Those who desire to trace the development and evolution of the present ACI Code from the initial efforts by the "Joint Committee" organized in 1904 and making its first report in 1914, are referred to an article, "Fifty Years of Development in Building Code Requirements for Reinforced Concrete," by Kerekes and Reid.* This paper gives a detailed account of the gradual changes in many important items considered along four channels of development (a) for the existence of a provision, (b) for a change in a provision, and (c) for the addition or deletion of a provision:

1. By analyses based upon the statics of internal forces.
2. By modifications of moment coefficients to provide practical expressions approximating probable theoretical values.
3. By recommendations of permissible unit stresses in flexure, compression, tension, shear, diagonal tension, bond, and bearing in terms of up-to-date knowledge of the physical properties of cement, aggregates, and steel acting individually or in combination.
4. By introduction of factors of safety to take into account the unpredictable elements of design as they are represented by materials, loads, structural elastic behavior, exposure to climatic variations and forces of nature, and the in-service demands upon the structure during its economic life.

Content of ACI Building Code

The Code covers the proper design and construction of ordinary buildings of reinforced concrete. In such structures as powerhouses, mill buildings, arches, tanks, reservoirs, and chimneys, just to mention a few typical instances, where specialization relates principally to the mechanics of design and details of construction, the provisions of this Code may be applied with the modifications necessary to suit the special conditions. The Code is written in such form that it may be incorporated verbatim or adopted by reference in a general building code; earlier editions have been widely used in this manner.

Among the subjects covered are:

1. General scope of the Code and a list of specifications of the American Society for Testing Materials referred to in the Code.
2. Materials and tests governing loading and criteria for acceptability, properties of cement, aggregates, steel and water, and storage of materials.
3. Concrete quality and allowable stresses including quality control, determination of concrete strength, proportions and consistency.
4. Mixing and placing of concrete including preparation of equipment and plan of deposit and requirements for conveying, curing, placing, and cold weather precautions.
5. Forms and details of construction including requirements for embedment of conduits, pipes, and other items in concrete.
6. General conditions governing design methods, loads, resistance to wind and earthquake.
7. Flexural computations in terms of general conditions and requirements of design, shrinkage and temperature reinforcement, concrete joist floor construction, and two-way systems with supports on four sides.
8. Shear and diagonal tension provisions by means of web reinforcement, including flat slabs, and in footings.
9. Bond and anchorage for flexural steel and for web reinforcement.

*ACI JOURNAL, Feb. 1954, *Proc.* V. 50, pp. 441-471.

10. Flat slabs with square or rectangular panels giving design procedures by elastic analysis and by empirical method.
11. Design and construction requirements for reinforced concrete columns and walls.
12. Design and construction requirements for footings.
13. Precast concrete design, construction, curing, storage, and transportation.

Appendix: Abstract of report on ultimate strength design of the ACI-ASCE joint committee on ultimate strength design.

How ACI Committee 318 functions

There are entirely too many users of building codes who believe that a group of committee members get together a few times and arbitrarily arrive at the provisions that are found in codes. So far as the work of ACI Committee 318 is concerned, such procedure is far from the actual manner of establishing revisions for a new issue of its building code.

Because the content of the Code covers almost every phase of knowledge and practice in reinforced concrete design and construction, it is necessary to assemble a large number of highly qualified members. There are, at this time, 37 members of the entire committee. For the work completed last year, the committee was divided into 14 subcommittees, each assigned to one of the chapters of the Code; in addition to the 12 chapters in ACI 318-51, two subcommittees were added—one to study code requirements for precast concrete and the other for prestressed concrete.

The personnel of each subcommittee carefully reviews the content of its assigned chapter to make revisions, additions, and deletions in terms of recent developments in design and construction and in terms of individual experiences coupled with a considerable amount of pertinent suggestions sent to the chairman of ACI Committee 318.

From time to time a meeting of the committee as a whole is called to review the preliminary progress reports of subcommittees. In this way the entire committee learns well in advance of final decisions what the proposals of subcommittees are likely to involve; and the subcommittees in their turn receive the benefit of counsel from the entire committee. For the past 5 years at each regional and annual convention the subcommittees deliberated during day sessions and then the entire committee met in the evening. Interested members of ACI were invited to attend and, to some extent, to participate in the discussions. During the past year, special meetings of the entire committee were held to prepare final proposals for the new Code. These final proposals for revision had to have a favorable letter ballot approval of the entire committee; where a member did not approve any item, he was requested to give his reasons for disagreement.

The final version of revisions in an ACI Standard have (1) to be referred for approval to the Standards Committee; (2) to be published in the JOURNAL at least one month before the annual convention; (3) to be favorably voted upon at the annual convention; and (4) to be approved by letter ballot of the entire membership of the Institute before the proposed revised version of the Building Code or other standard can be declared as a new Standard of ACI.

From the foregoing brief description of how ACI 318-56 was processed, it should be clear that every possible precaution is taken to maintain the usefulness, accuracy, authoritativeness, and prestige of the Code. In the case of ACI 318-56, there were about 600 members at the afternoon session when the revisions were considered at the Philadelphia convention of the Institute. After 3 hours of presentation by each subcommittee chairman and discussion from the floor, ACI 318-56 was unanimously approved by those present. Later the letter ballot of those of the entire membership who voted overwhelmingly approved the proposed new Code.

Philosophy of ACI Building Code

"Building Code Requirements for Reinforced Concrete (ACI 318-56)" is not a compilation of rule-of-thumb short cuts to substitute for intelligent design and construction of reinforced concrete structures. The Code should not be used as though it were a "bible" whose contents tolerate no interpretative applications, or as though it were a "handbook" from the pages of which specific answers could be plucked for each and every problem that may confront the designer. The Building Code is not a substitute for a thorough knowledge of the facts and principles that control the properties of materials, the elastic behavior of structures, the theoretical analyses of structural systems, and the quality of recognized construction methods and procedures. The Building Code, above all, is no substitute for that judgment which must be exercised in selecting and combining those facts and principles which apply in each design and construction.

What then is the function of a building code?

A building code is a document that presents in concise form a guide to design and construction. Each item in the code is there because time and experience have demonstrated that its judicious application in appropriate cases has led to satisfactory and dependable results. Therefore, it is quite likely to prove dependable and satisfactory in future applications only when coupled with the designer's thorough understanding and insight of the behavior of his structure. A building code is a guide to *minimum* standards only when it is employed in instances of the more usual or common types or systems of structures. Nevertheless, when a careful and thorough structural analysis accompanies the unusual or complex structural system, a building code can still be a useful guide in the design and construction of the integral parts of the structure. At no time or occasion are the provisions of a building code a substitute for intelligent analysis, shrewd interpretation, or balanced judgment; with these as a preliminary, a building code can be of tremendous value both to the inexperienced and experienced designer and builder because the code brings to him the cumulative counsel of many authorities representing a wide variety of experience beyond the possible attainment of any one person.

Relation of the building code to teaching and to practice

Engineering is an art as well as a science. As a rule the "science" of any area of knowledge is theoretical and lends itself to textbook presentation

and development. On the other hand, the "art" of any area of knowledge depends in many cases upon intangibles and upon individual interpretations usually and importantly based on experience and upon a knowledge of what is available and how it can be done from a practical viewpoint. Thus, a building code is a guide to the "art" of reinforced concrete design and construction to be used only when and after the "science" of reinforced concrete design and construction has been employed. A building code can be, and is, an indispensable adjunct to a textbook on reinforced concrete design. An alert and intelligent student should be able to design reinforced concrete structures without the use of the building code. But he would consume much time and effort in arriving at solutions to the many questions that would confront him. And the labor would in all probability result in a unique and possibly uneconomical design. The use of the building code as a guide enables the student to gain accelerated experience by direct application instead of time consuming trial and error approaches. This should not imply that there is removed from the student all stimulation for creative thinking. There still remain to him many channels for creative thinking. The teacher should use the building code, not as a mechanism for turning out stereotyped designs, but rather as a vehicle for discussing why, how, and when the code requirements came into acceptance and general use. The teacher certainly should encourage his students to challenge the requirements in codes; and he should make known to the students that code requirements are subject to change as soon as more accurate theory and more dependable methods are established.

As to the relation of the building code to practice, the important viewpoints have been presented in the discussion on the philosophy of the ACI Building Code. The important decisions that must be made by the experienced man in practice are to determine the extent to which the conditions of his design or construction are identical with the building code provisions. The experienced practical designer should take into account as many of the forces and influences as he can determine for the particular structure he has under consideration. His analysis of especially unusual structural types and systems should anticipate every possible elastic and geometric action. Then he can, to good advantage, be guided by appropriate and related provisions of the building code. But the provisions of a building code are not intended to make up for lack in theoretical understanding, for the judgment to benefit by the experiences of others, and for the skill in anticipating the behavior of related portions of a complex structural system. Once the action of each component part of a complex structure is analyzed, and the interaction of the component parts is determined, the building code provisions can be utilized to good advantage and with a reasonable amount of assurance.

SUMMARY OF CODE REVISIONS

In this summary there will be discussed only the highlights of the major changes in ACI 318-56. Specifically these are: ultimate strength design,

design of flat slabs, revisions in column design, new provisions for shear, inclusion of provisions for radiant heat pipes in structural concrete slabs, load tests, air entrainment, mix proportioning, quality control, and precast concrete.

Ultimate strength design

Chapter 6, Section 601(b) recognizes for the first time that "The ultimate strength method of design may be used for the design of reinforced concrete members."

Realizing that most users of the ACI Building Code would not have ready access to pertinent literature and realizing further that designers should be guided by what may be considered the most authoritative current treatment of the subject, the members of ACI Committee 318 have included in the appendix of the Code a ready reference in the form of an *abstract* of the report of the ACI-ASCE joint committee on ultimate strength design. For reliable basic information the designer should add to his library Paper No. 809, *Proceedings*, ASCE, V. 81, Oct. 1955, or "Ultimate Strength Design," ACI JOURNAL, Jan. 1956, *Proc.* V. 52, pp. 505-524; and also, "Guide for Ultimate Strength Design of Reinforced Concrete," by Charles S. Whitney and Edward Cohen (ACI JOURNAL, Nov. 1956, *Proc.* V. 53, pp. 455-490.

The term "ultimate strength design" indicates a method of design based on the ultimate strength of a reinforced cross section in simple bending, and combined bending and axial load on the basis of inelastic action.

The recommendations are confined to the design of sections. It is assumed that external moments and forces acting in a structure will be determined by the theory of elastic frames. With the specified load factors, stresses under service loads will remain within safe limits, and an ample factor of safety is provided against an increase in live load beyond that assumed in design; strains under service load will not be so large as to cause excessive cracking.

Design of flat slabs

The revised version of Chapter 10 on flat slabs has a greater practical value since it gives requirements in more detail, particularly when the design is made by empirical method, and includes flat plate and slab band systems. The division in scope between shallow two-way beams with two-way slabs and flat slab "slab bands" is defined.

Either of two methods of analysis is permitted: (1) design by elastic analysis: Chapter 10, Section 1003; (2) design by empirical method: Chapter 10, Section 1004.

Design by elastic analysis—The following assumptions may be used and all sections shall be proportioned for the moments and shears thus obtained.

1. The structure is considered as a number of longitudinal and transverse bents consisting of columns or supports and strips of supported slabs.
2. These bents may be analyzed in their entirety; or, each floor thereof may be analyzed separately with its adjacent columns as they occur above and below, the columns being assumed fixed at their remote ends.

3. The joints between columns and slabs may be considered rigid extending in the slabs from the center of column to edge of the capital and in the column from top of slab to bottom of capital.

4. Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and shear.

5. The moment of inertia of the slab or columns at any cross section may be assumed to be that of the cross section of the concrete.

6. Where the load to be supported is definitely known, the structure shall be analyzed for that load.

Bending at critical sections across the slabs of each bent may be apportioned between column-strip and middle-strip, as given in Table 1003(c). For design purposes, any of these percentages may be varied by not more than 10 percent of its value, but their sum for the full panel width shall not be reduced.

Design by empirical method—The designer shall conform to all of the limitations on continuity and dimensions given in Chapter 10, Section 1004. General limitations are:

1. The construction shall consist of at least three continuous panels in each direction.
2. The ratio of length to width of panels shall not exceed 1.33.
3. The grid pattern shall consist of approximately rectangular panels.
4. The calculated lateral force moments from wind or earthquake may be combined with the critical moments as determined by the empirical method and distributed between the column and middle strips for structures not exceeding 125 ft in height with story heights not exceeding 12 ft 6 in.

To facilitate computations Table 1004(f) is provided to give moments in flat slab panels in percentages of M_o. Also, two diagrams, Fig. 1004(f)a and 1004(f)b, are provided to enable the designer to visualize the moments in flat slab panels in percentages of M_o without drop panels and with drop panels.

In addition to the requirements of Section 1002(d), reinforcement shall have the minimum lengths given in Tables 1004(g)1 and 1004(g)2. Where adjacent spans are unequal, the extension of negative reinforcement on each side of the column center line as prescribed in Table 1004(g)1 shall be based on the requirements of the longer span. To assist the designer, Fig. 1004(g) presents the diagramatic schemes for visualizing the requirements for the minimum length of flat slab reinforcement.

Reinforced concrete columns and walls

The formula for allowable load on concrete-filled pipe columns has been changed to utilize the latest provisions of the AISC code for unfilled pipe columns. The allowable loads shall be determined by the formula

$$P = 0.25 f_c' \left(1 - 0.000025 \frac{h^2}{K_c^2} \right) A_c + f_r' A_s$$

The value of f_r' shall be given by the formula

$$f_r' = 17{,}000 - 0.485\frac{h^2}{K_s^2}$$

when the pipe has a yield strength of at least 33,000 psi and an h/K_s ratio equal to or less than 120.

Section 1109, "Columns Subjected to Axial Load and Bending," has been completely revised and rewritten. The former equations in ACI 318-51 and its predecessors are replaced by simpler, well established equations involving the following terms:

F_a = nominal allowable axial unit stress ($0.225f_c' + f_sp_g$) for spiral columns and 0.80 of this value for tied columns
F_b = allowable bending unit stress that would be permitted if bending stress only existed
f_a = nominal axial unit stress = axial load divided by area of member, A_g
f_b = bending unit stress (actual) = bending moment divided by section modulus of member

Members subject to an axial load and bending in one principal plane, but with the ratio of eccentricity to depth no greater than 2/3, shall be so proportioned that

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ does not exceed unity}$$

When bending is oblique to the principal axes, the formula becomes

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \text{ does not exceed unity}$$

where f_{bx} and f_{by} are the bending moment components about the x and y principal axes divided by the section modulus of the transformed section relative to the respective axes, provided that the ratio e/t is no greater than 2/3 in either direction.

A suggested formula for computing preliminary sections when columns carry both axial load and bending is

$$P = N\left(1 + \frac{Be}{t}\right)$$

in which N equals the axial load and B is a trial factor which may be taken as 3 to 3½ for rectangular tied columns; for circular spiral columns, the value of B from 5 to 6 may be used.

Shear, diagonal tension, bond, and anchorage

The principal changes made are in Section 801(d) and in the addition of Section 801(e). Wherever the value of the shearing unit stress computed by

$$v = \frac{V}{bjd}$$

exceeds the shearing unit stress v_c permitted on the concrete of an unreinforced web (see Section 305), web reinforcement shall be provided to carry the excess. Such reinforcement shall also be provided for a distance equal to the depth, d, of the member beyond the point theoretically required.

Where continuous or restrained beams or frames do not have a slab so cast as to provide T-beam action, the following provisions shall apply. Web reinforcement shall be provided from the support to a point beyond the extreme position of the point of inflection a distance equal either to 1/16 of the clear span or the depth of the member, whichever is greater, even though the shearing unit stress does not exceed v_c. Such reinforcement shall be designed to carry at least two-thirds of the total shear at the section. Web reinforcement shall be provided sufficient to carry at least two-thirds of the total shear at a section in which there is negative reinforcement.

The computation of bond stress in beams is still by the formula

$$u = \frac{V}{\sum ojd}$$

in which V is the shear in flexural members in which the tensile reinforcement is parallel to the compression face and Σo is taken as the perimeter of all effective bars crossing the section on the tension side. Bent-up bars that are not more than $d/3$ from the level of the main longitudinal reinforcement may be included. Critical sections occur at the face of the support, at each point where tension bars terminate within a span, and at the point of inflection.

Bond shall be similarly computed on compressive reinforcement, but the shear used in computing the bond shall be reduced in the ratio of the compressive force assumed in the bars to the total compressive force in the bars at the bond stress in Table 305(a).

To the anchorage requirements are added the following: At least one-third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance of sufficient length to develop by bond ½ the allowable stress in such bars, not less than 1/16 of the clear span length, or not less than the depth of the member, whichever is greater. The tension in any bar at any section must be properly developed on each side of the section by hook, lap, or embedment (see Section 906). If preferred, the bar may be bent across the web at an angle of not less than 15 deg with the longitudinal portion of the bar and be made continuous with reinforcement which resists moment of opposite sign.

Other important revisions

For quite some time few changes have been made in Chapters 2 through 5 even though there have been continuous new developments in materials and tests, in concrete quality and allowable stresses, in mixing and placing concrete, and in forms and details of construction. ACI 318-56 contains significant changes that reflect concrete technology and construction in terms of current knowledge and practice.

In the chapter on concrete quality and allowable stresses the suggested water-cement ratios have been adjusted to take advantage of air-entrained concrete and of the recommendations provided by ACI Committee 613—Recommended Practice for Proportioning Concrete Mixes. Two methods for determining the strength of concrete are provided: Method 1—without preliminary tests [Table 302(a) gives permissible water-cement ratios for concrete]; and Method 2—with preliminary tests. These tests shall be made in accordance with the procedure given in the appendix to "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)".

In Table 305(a), "Allowable Unit Stresses in Concrete," a new column is inserted for $f_c' = 5000$ psi, the minimum specified compressive strength at 28 days. Instead of allowing $0.12f_c'$ for v for beams with properly designed web reinforcement, ACI 318-56 allows $0.08f_c'$ for v for beams with longitudinal bars and with either stirrups *or* properly located bent bars; the Code allows $0.12f_c'$ for v for beams with longitudinal bars and a combination of stirrups and bent bars (the latter bent up suitably to carry at least $0.04f_c'$).

Mixing of concrete is tied in with "Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)." Approved procedures for cold weather requirements are tied in with "Recommended Practice for Winter Concreting (ACI 604-56)."

In Chapter 5, "Forms and Details of Construction," Section 503 dealing with conduits, pipes, etc., embedded in concrete has been completely rewritten. The committee now presents code requirements which permit the embedment of pipes for radiant heating and other uses provided the provisions of Section 503 are followed.

SUMMARY

The statement that the Building Code is merely a guide to minimum requirements applicable in its entirety only to common structures is worth repetition for emphasis. Statements that the failure of some unusual detail, element, or structure reflects a deficiency in the Code are meaningless, especially when extrapolating outside the range of common experience requires professional judgment to determine how much to exceed the minimum requirements furnished by the guiding code.

TRENDS FOR THE FUTURE

It can be safely predicted that the next revision of the ACI Building Code will contain considerable guidance in the precast field as practical experience is accumulating fast and theory is unchanged. Recognition of prestressed concrete is also to be considered for inclusion especially since the ACI-ASCE joint committee on prestressed concrete has released its preliminary report; much remains to be learned, both in practical experience and in theory of prestressed concrete. Extensive tests on shear are being evaluated so that we may develop reliable methods of predicting shear strength which would allow us to design for a uniform safety factor against shear as well as flexural failure.

As the retiring chairman of ACI Committee 318, I fully appreciate the task involved in revising the Code. It seems safe to predict plenty of important and hard work ahead for the committee to complete the next revision by, say, 1960.

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Title No. 54-12

Ultimate strength design provisions—One of a series explaining sections of the ACI Building Code

Explanatory Notes on Appendix (Ultimate Strength Design) to "Building Code Requirements for Reinforced Concrete (ACI 318-56)"*

Reported by ACI COMMITTEE 318

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SYNOPSIS

A series of questions and answers on the appendix ("Abstract of Report of ACI-ASCE Joint Committee on Ultimate Strength Design")† to the latest ACI Building Code, ACI 318-56.

In response to requests for elaboration of certain sections of the Appendix on ultimate strength design in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," Committee 318 has grouped the following questions so that all readers may have the benefit of these explanations.

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Prepared as a part of the work of ACI Committee 318, Standard Building Code. The report was approved by the committee in form and substance as here submitted.

†The Appendix appears on pp. 981-986 of the Code (ACI 318-56), ACI JOURNAL, May 1956, *Proc.* V. 52; the full report, of which the Appendix is an abstract, was published in *Proceedings*, ASCE, V. 81, Paper No. 809, Oct. 1955; see also ACI JOURNAL, Jan. 1956, *Proc.* V. 52, pp. 505-524.

1—Section A604 expresses ultimate strength capacity, *U*, in terms of the effects of live load plus impact, *L*, and basic load, *B*. The definition in A600 states "*B* = effect of basic load consisting of dead load plus volume change due to creep, elastic action, shrinkage, and temperature." How is this term *B* to be evaluated?

For the design of reinforced concrete arches, rigid frames, and similar structures in which it is normal practice to take into account the effect of creep, elastic action, shrinkage, temperature, and settlement when designing by the working stress method, those same effects should be included with the effect of dead load to give the basic load effect, *B*, in ultimate strength design.

For other structures such as building frames of the usual types of construction, spans, and story heights, where it is not customary to consider the effects of creep, shrinkage, etc., when designing by the working stress method, it is not necessary to consider them in ultimate strength design. In other words, for most structures and conditions the basic load effect *B* may be considered to be the dead load effect only.

2—Section A604(a) gives two formulas [Eq. (I) and (II)] for determining the ultimate load. Which is to be used?

In Eq. (I), the basic load (which can be determined with a good degree of precision) is multiplied by a factor 1.2, while the live load (which at best is only a reasonable interpretation of the probable occupancy of the structure and which can vary with the life of the structure) is multiplied by a factor 2.4. Therefore, the live load has twice the significance in arriving at the ultimate capacity of a member as does the basic load. If the live load is several times as great as the basic load, the net result will be a relatively heavy construction. If the live load is equal to or less than the basic load, some other criterion must be used, or the structure could become too light. Eq. (II) was included to prevent the ultimate capacity being less than *K* times the sum of the basic and live loads, the factor *K* for beams and girders being 1.8. Eq (I) will govern when the live load equals or exceeds the basic load and Eq. (II) when the live load is less than the basic load. The smallest ultimate load will be 1.8 times the sum of the basic and the live loads.

Specifically, then, both formulas should be applied and the one which requires the greater capacity is used.

3—Section A602(a) states that the Code proper applies except where otherwise provided in the Appendix. Which provisions of the Code do apply and which do not?

The Appendix states that those of its provisions which modify requirements of the Code proper supersede those of the original Code. Therefore, all the requirements that are not specifically replaced or modified are still in full force. This applies throughout the entire Code.

4—The Appendix puts no limitations on the width of beam tees; should they be the 8t of the original Code or the 6t of the ACI-ASCE joint committee report on ultimate strength design?

Under ultimate strength design the width of tee is seldom a factor. It is difficult to put in sufficient reinforcing steel to make the width of flange critical. It is a specific requirement that the 8t of the original Code be a top limit (possibly the 6t of the ACI-ASCE joint committee report might be a better limit, but those working with ultimate strength design soon find that this is a theoretical question because of the lack of need for additional compressive width).

5—Does Section A602(a) mean that shear and bond are to be designed by the original Code or is it possible to take care of them under the Appendix?

Joint ACI-ASCE Committee 326 has, for some years, been making a thorough study of the entire subject of shear and diagonal tension, planning and seeing through a whole group of tests to shed more light upon this subject, but it is not ready to formulate rules from these researches.

For the time being, proportion members for shear and bond for service loads in accordance with Chapters 8 and 9 of the Code. If desired, actual calculations can be simplified by considering the ultimate shear and bond strengths as being equal to the allowable working stresses for shear and bond obtained from Table 305(a) multiplied by the ratio of U (obtained from the formulas in Section A604) to the sum of basic load plus live load but not less than 2.

The researches under way should soon advance to a point where it will be possible to amplify the requirements of the Appendix to include an ultimate strength method of design for shear and bond.

Ultimate strength design recognizes steels with high yield points. Tests suggest that allowable diagonal tension and bond values might well be lowered with increased steel stress, but, until the tests produce specific answers, it is suggested that when f_y exceeds 40,000 psi in the computations, the allowable shear and bond stresses be reduced to 85 percent of the values given in Table 305(a).

6—Section A602(e) requires a check for the deflection of shallow members, that is, members where p exceeds $0.18f_c'/f_y$. How is this done?

When f_c' is equal to 3000 psi and f_y is 40,000 psi, then $p = 0.18f_c'/f_y = 0.0135$, which is almost exactly the same as 0.0136 which is obtained for balanced reinforcement under the working stress method. This is equivalent to saying that when the steel percentage is not higher than balanced reinforcement under the working stress method, no check of deflections need be made other than would have been made for shallow slabs or beams under the working

stress method. When more steel is used in a member and the depth is made shallower, a check upon deflection must be made.

The determination of the deflection of a reinforced concrete beam is complicated by several factors: (1) a nonhomogeneous material, (2) the monolithic character of structure developing restraining moments at ends of members, (3) shrinkage of the concrete during the hardening process, and (4) creep with age. As regards nonhomogeneous character, satisfactory results are obtained by using the properties of the gross cross section of the concrete, omitting the reinforcing steel.* (For ordinary T-beams I_c can be taken as $1\frac{3}{4}$ to $2\frac{1}{2}$ times the moment of inertia of the stem, or charts are available giving approximate values of I_c for T-beams.)

The end restraining moments are readily taken care of by breaking the problem into three parts: the deflection due to transverse loads assuming freely supported end conditions, and the deflection due to a separately applied moment on either end of a freely supported beam.

Shrinkage and creep cause additional deflection for which elaborate mathematical studies have been made, but practical results are obtained by increasing the short-time elastic deflection to about $2\frac{1}{2}$ times that value.

For example, what is the probable short-time deflection of a continuous 4-in. concrete floor slab reinforced with #6 bars at 6 in. center-to-center carrying a total load of 250 psf on a span of 15 ft, $f_c' = 3000$ psi, $f_y = 50{,}000$ psi, $n = 10$, if the moment at the left end is 30 in.-kips and at the right end 35 in.-kips?

The uniform load produces downward deflection, maximum near midspan; left end moment produces upward deflection maximum left of center; right end moment produces upward deflection maximum right of center. It is sufficiently accurate to combine the three maxima for center deflection.

$$I_c = \frac{bd^3}{12} = \frac{12 \times 4^3}{12} = 64$$

$$\Delta \text{ uniform} = \frac{WL^3}{76.8EI} = \frac{250 \times 15^4 \times 1728}{76.8 \times 3{,}000{,}000 \times 64} = +1.48 \text{ in.}$$

$$\Delta \text{ 30 in.-kips} = \frac{3M_L L^2}{48EI} = \frac{-3 \times 30{,}000 \times 15^2 \times 144}{48 \times 3{,}000{,}000 \times 64} = -0.32 \text{ in.}$$

$$\Delta \text{ 35 in.-kips} = \frac{3M_R L^2}{48EI} = \frac{-3 \times 35{,}000 \times 15^2 \times 144}{48 \times 3{,}000{,}000 \times 64} = \frac{-0.37 \text{ in.}}{+0.79 \text{ in.}}$$

If the applied loads (live, dead, and moments) remain in place for several years, what final deflection is likely to result?

This illustration does not lend itself so readily to exact mathematical analysis. Recent tests of continuous beams indicate that the long-time actual deflection is at least

*This method is used for illustration because of its simplicity and because the results obtained are sufficiently indicative for the control of designs. There are advocates for the use of the transformed section and of a combination of transformed and gross sections, because the bars, compression steel in particular, have a pronounced effect on the amount of long-time deflection.

2 or 2½ times the short-time elastic deflection. Then $\Delta = 2\frac{1}{2} \times 0.79 = 1.98$ in. (This value may even increase in beams without compressive reinforcement, with rich concrete, and with highly stressed steel.)

What limitation should be placed upon deflection? This depends upon the use to which the structure is to be put. Any floor which is supporting partitions or any ceiling which has tightly fitted casework below should certainly have a definite limit and $L/360$ is as much deflection as would seem desirable for this sort of occupancy.

Roofs which are freestanding and have no casework or other items on the underside could stand considerably more deflection before any unsightly or damaging results develop. Until experience and time have set better standards, it is suggested that even in those cases where excessive deflection does not affect any other parts of the structure, it is wise to limit the deflection to twice $L/360$ or $L/180$. This is a region in which judgment and experience are required.

7—Section A603(e) states that the stress in tensile and compressive reinforcement at ultimate load shall not be assumed greater than the yield point or 60,000 psi, whichever is smaller. Section 105 refers to the specifications of the American Society for Testing Materials and declares them to be a part of the Code the same as if fully set forth. Must the designer select steels which conform to the ASTM specifications?

The ASTM specifications are definitely a part of this Building Code. It is the intent of the American Concrete Institute to recognize and abide by the ASTM specifications. ASTM limits the minimum f_y to 40,000 psi for intermediate-grade and 50,000 psi for hard-grade and rail-steel deformed bars, which are then the values to use for standard grades of reinforcing steel in ultimate strength design. Since there are alloy steels and prestressing wires with higher yield points, the ACI-ASCE joint committee on ultimate strength design and Committee 318 considered the possibility of ASTM adopting a specification for reinforcing bars of higher yield point and recommended for the present an upper limit of 60,000 psi for the yield point to be used in design.

A word of caution. Designers have been using intermediate-grade and rail-steel deformed reinforcing bars and the mills have been supplying them according to the ASTM specifications. Certainly when a designer deviates from the commonly specified type of material, he assumes a greater responsibility for testing or otherwise assuring himself that the material actually delivered on the job is the type of material that he wishes in his structure.

8—Section A605(c), Eq. (A2), limits p to $0.40 f_c'/f_y$; what is the reason for this limitation?

Under ultimate strength design balanced reinforcement is considerably above that of the working stress method (perhaps 0.03 or more), so that greater percentages of reinforcing steel can be used effectively where the depth of members is controlled by architectural considerations. The limitation of $p = 0.40 f_c'/f_y$ is provided for members in pure bending in ultimate strength design so that the ultimate strength of a member will always be controlled by the steel and not by the concrete. The practical limit from an economic standpoint would be considerably less than given by this formula. Attention is also called to the need for determining deflections when high percentages of reinforcement are used.

9—Section A603(c) states that any form of stress block within reason, whether rectangular, trapezoidal, parabolic, or combination can be used. How much effect will this have on the final design?

The basic philosophy of ultimate strength design as discussed in Question 10 shows that the shape of the stress block is relatively unimportant. Its total volume must balance the tension in the steel and the location of its centroid is determined empirically, so the exact shape is of little importance to the practical designer. The researcher may be somewhat interested, but if the total compressive force in a flexural member can not change and its centroid can not move, its distribution throughout the cross section is at best an academic question.

10—Designs made under the Code proper and those made by the Appendix may differ somewhat. What are the reasons for this?

A stress block such as is proposed in ultimate strength design was one of the first approaches to the determination of internal stresses in reinforced concrete members. It was displaced by the elastic theory which has been developing into extended mathematical determinations until formulas have departed from the reality of the behavior of reinforced concrete structures. The assumptions upon which the elastic theory is based are not entirely reliable throughout the full range of action of reinforced concrete members.

It has long been recognized that stress is not proportional to strain as failure is approached; stress distribution follows more nearly the familiar parabolic stress-strain curve.

The modulus of elasticity of the concrete does not have a constant value. It is not always the same for various batches of concrete using the same aggregates; it changes with humidity, age, and with stress. Creep enters the problem. Any analysis made upon an assumption of a constant modulus of elasticity falls far short of the facts.

Except at low percentages of reinforcement, beams designed with steel stressed 50 percent of the yield strength and concrete stressed 45 percent of the ultimate compressive strength of cylinders may not fail at loads indicated from these working stresses. The elastic method of computation may not accurately predict the actual capacity of test specimens.

This does not mean that safe structures are not obtained by the working stress method. The Code has been adjusted by experience and judgment so that satisfactory and safe structures have been built by a multitude of designers all across the country over a long period. However, the factors of safety of these different structures are not at all uniform, are not even uniform throughout any one structure or throughout any one part of that structure. The only way to determine how much such factors vary is to compute the capacity by ultimate strength design and compare that value with the one computed by the working stress method.

The ultimate strength method, on the contrary, is not a philosophy or theory at all; it is obtained directly from a statistical study of the results of all known tests of reinforced concrete members. The fundamental theory of bending assumes internal compressive and tensile forces and, with the reinforcing steel concentrated, the tensile force can be approximated in size and location. Since tension and compression are equal, this leaves the location of the centroid of compression as a variable factor. It is, therefore, possible to study all tests and determine the amount of compressive force and arm to maintain static equilibrium. The result of such study is the basis of ultimate strength design. Obtained in this manner, ultimate strength design can predict closely the capacity of any member.

In the 1936 revision of the Code, the basis for the column formulas was changed from a linear stress-strain relationship to an ultimate strength basis for better agreement with test results. At that time, the Code left bending in the working stress category, so that a member subjected to combined bending and direct stress was a hybrid, its bending resistance being approximated by the working stress method and its compression by the ultimate strength method. Utilizing ultimate strength design throughout a structure is a much more accurate and logical procedure.

Ultimate strength design is considerably used throughout the world. Many countries either recognize it or use it as the only method. It is simpler to apply, gives much more consistent results, and affords the designer much better opportunity to estimate what overload factors he has in appraising existing structures. Some good ideas are contained in Chapter 2 of Pier Luigi Nervi's book, *Structures*, as translated by Giuseppina and Mario Salvadori, which emphasizes some of the difficulties and uncertainties of elastic analysis and some of the advantages of ultimate strength design.

11—Is the use of arbitrary moment coefficients permissible in ultimate strength design?

Yes, Section A602(a) says the provisions of the Code proper apply, and the same methods of determining external moments are used in ultimate strength design and design by working stresses. However, for even those load and span limitations for which the Code permits the use of arbitrary coefficients, in the case of heavy live loads, the designer can readily see that $2.4L$ will have a greater effect than $1.2B$, so that, in cases of considerable variation in span, moment computations and locations of points of contraflexure might be more carefully verified by exact analyses.

Discussion of this report should reach ACI headquarters in triplicate by Dec. 1, 1957, for publication in the March 1958 JOURNAL.

Laboratory Tests of Portland Blast-Furnace Slag Cements*

By BRYANT MATHER†

SYNOPSIS

Laboratory tests of samples of portland blast-furnace slag cements, and of the blast-furnace slags and portland cement clinkers used in their manufacture, obtained from each of the mills making it in the United States in 1955, indicate that the then current federal and ASTM specifications provide adequate assurance of performance at least equal to that insured of Type I portland cements by applicable specifications. Portland blast-furnace slag cements, meeting the specifications, frequently have low enough heats of hydration to meet the optional heat of hydration requirement for Type II portland cement of the federal specification.

Specifications for portland blast-furnace slag cements do not insure that they will have moderate sulfate resistance, as required of Type II portland cements. The performance of an experimental portland blast-furnace slag cement containing more magnesia than permitted by the specifications, made with a high-magnesia (9.6 percent) slag, was not found to have been adversely affected. Mortar-bar tests suggest that the presence of the slag in the cement acts to keep expansion due to alkali-aggregate reaction from becoming excessive, even when a highly reactive aggregate is used and the cement contains more than 0.6 percent alkalies calculated as sodium oxide.

INTRODUCTION

The use of blast-furnace slag as a cementitious material dates at least from 1774 when Loriot made a mortar using blast-furnace slag and slaked lime. Portland blast-furnace slag cements have been widely used in Europe since about 1855. In Europe, two grades are distinguished: one, *eisenportland*, containing up to 30 percent slag, and another, *hochofen*, containing from 30 to 70 percent slag. However, since 1916, the Germans have used a single specification for both of these grades and for portland cement.[1] An investigation of the manufacture and properties of slag ("pozzolan") cement was made by the Corps of Engineers in 1900.[2] Among the conclusions of this investigation[3] was: "The cement is well adapted for use in sea water . . . and in the interior of heavy masses of masonry or concrete." In 1914 about 70,000 bbl of slag cement were made in the United States.[4] In the past few years a number of portland cement producers have begun the manufacture of portland blast-furnace slag cement. The first edition of the ASTM "Speci-

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fications for Portland Blast-Furnace Slag Cement" (ASTM C 205)[5] was adopted in 1946, and revised in 1947, 1948, 1951, 1953, and 1956. The federal specification for "Cements, Portland, Blast-Furnace Slag" (SS-C-197) was issued in 1952.[6]

At the time these tests were begun, portland blast-furnace slag cement was being made commercially in the United States by six manufacturers at seven mills and experimentally by a seventh company. Subsequently, another manufacturer has begun commercial production at two mills and others have indicated their intention to begin production in the relatively near future. Samples of portland blast-furnace slag cement, slag, and portland cement clinker were obtained from each of the mills.

The purpose of these tests was to develop data on the properties and performance of these samples of portland blast-furnace slag cement and to determine the additional requirements, if any, that would permit its use as a substitute for Type II portland cement in Corps of Engineers concrete construction. It was assumed that the properties and performance of any given sample of portland blast-furnace slag cement would be determined by the properties of the slag and portland cement clinker used in its manufacture, their proportions, and the fineness to which the resulting product was ground; therefore, samples of the slag and clinker as well as of the portland blast-furnace slag cement, and information on the proportions used, were obtained. Six sets of samples of commercial Type IS portland blast-furnace slag cements, slags, and clinkers are designated by serial numbers 337, 338, 339, 340, 341, and 345; a Type IS-A is designated 342; the experimental, high-MgO, material is designated 336; and the reference Type II portland is designated 330.

TABLE 1—CHEMICAL DATA ON SLAGS AND PROPORTIONS OF SLAGS USED IN MANUFACTURE OF PORTLAND BLAST-FURNACE SLAG CEMENTS*

Test data	Serial No.							
	336	345	338	341	339	340	337	342
Moisture, percent	1.35	0.00	0.06	0.42	0.13	0.10	0.08	0.13
CaO, percent	39.50	42.48	43.46	43.47	44.53	46.27	46.42	47.10
SiO_2, percent	32.21	37.80	32.11	33.58	37.25	35.92	35.72	33.39
Al_2O_3, percent	10.56	11.34	12.05	12.35	11.70	10.71	9.76	12.22
MgO, percent	9.65	5.61	5.85	2.04	3.40	3.93	5.28	3.37
S, percent	1.49	0.77	1.68	0.98	1.51	1.29	1.10	1.11
Fe_2O_3, percent	1.05	0.41	1.37	0.43	2.07	0.42	0.38	1.23
Mn_3O_4, percent	0.23	0.81	1.25	0.35	0.59	1.19	1.32	0.91
SO_3, percent	0.19	0.06	0.49	0.23	0.01	0.00	0.00	0.01
Na_2O, percent	0.64	0.25	0.23	0.15	0.25	0.23	0.21	0.24
K_2O, percent	0.67	0.75	0.43	0.20	0.33	0.38	0.24	0.35
Total as Na_2O, percent	1.08	0.74	0.51	0.28	0.47	0.48	0.37	0.47
Ignition loss, percent†	2.68	−0.85	−1.01	3.17	−2.99	−1.15	−1.19	−1.16
Insoluble residue, percent	1.44	0.29	1.12	0.64	1.70	0.75	0.44	0.60
Hydraulic index, F (1)	1.34	1.14	1.33	1.19	1.15	1.25	1.30	1.30
Data from manufacturer								
Percent slag used	62	42	45.0	25	40	32	43.0	40
Percent clinker used	34	53	51.3	‡	‡	62	52.7	60
Percent gypsum used	4	5	3.7	‡	‡	6	4.3	‡

*Arranged in order of increasing CaO content.
†Negative values indicate a gain in weight on ignition; such gain is believed due to oxidation of sulfide sulfur and iron or manganese constituents.
‡No data furnished.

SLAGS

Chemical data

Chemical data on the eight samples of slag are given in Table 1. Keil[7] notes that there are three "basicity ratios" that are used for the judging of melting conditions in the blast furnace:

$$P_1 = \frac{CaO}{SiO_2} \qquad P_2 = \frac{CaO + MgO}{SiO_2} \qquad P_3 = \frac{CaO + MgO}{SiO_2 + Al_2O_3}$$

He also notes the various attempts that have been made to represent the complicated relationships between chemical composition and hydraulic properties of slag by the use of formulae:

$$\frac{CaO + MgO + 1/3\ Al_2O_3}{SiO_2 + 2/3\ Al_2O_3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\frac{CaO + MgO + Al_2O_3}{SiO_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$\frac{CaO + CaS + 1/2\ MgO + Al_2O_3}{SiO_2 + MnO} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Formula (1) appears in current federal and ASTM specifications for portland blast-furnace slag cements (see Tables 4 and 14). All the slag samples have values for this ratio in the range 1.14 to 1.34 (Tables 1 and 4). This ratio is that formerly contained in German cement standards and was found in Keil's work to vary from 0.85 to 1.35. Studies by L. R. Forbrich (unpublished) tend to suggest that although none of the ratios provides a very precise index of hydraulic activity, the others, especially Formula (2), are somewhat more closely related to such activity than Formula (1). Selected ratios for the eight slags are listed below:

Slag No.	Ratios				
	CaO/SiO_2* (P_1)	SiO_2/Al_2O_3*	Formula (1)	Formula (2)	Formula (3)*
336	1.23	3.05	1.34	1.85	1.72
337	1.30	3.66	1.30	1.72	1.61
338	1.35	2.66	1.33	1.91	1.78
339	1.20	3.18	1.15	1.60	1.55
340	1.29	3.35	1.25	1.70	1.61
341	1.29	2.72	1.19	1.72	1.69
342	1.41	2.73	1.30	1.88	1.80
345	1.12	3.33	1.14	1.57	1.48

*According to Keil[7] slags with values of Formula (3) less than 1.5 are of medium hydraulic properties, from 1.5-1.9 good, and greater than 1.9 very good. He also quotes Blondiau as having found that suitable slags have a CaO/SiO_2 ratio of 1.45-1.54 and a SiO_2/Al_2O_3 ratio of between 1.8 and 1.9.

Petrographic and x-ray diffraction data

The eight samples of slag were examined using microscope techniques and by x-ray diffraction. Both of these procedures contributed data that

enable positive identification of the crystalline constituents. If some one crystalline constituent had been found present in appreciable amounts in all of the slags, it might then have been possible to use the x-ray diffraction record of this constituent to estimate the glass content of the slags. However, such was not the case.

The limitation of 5.0 percent on the MgO content of portland blast-furnace slag cements, now included in federal and ASTM specifications, has been the subject of considerable discussion. It has been stated that cements made with slags of such high MgO content that the resulting cement contains as much as 10 percent MgO are entirely sound and have given satisfactory service in Europe and that such performance should be expected since the composition and manner of cooling of blast-furnace slags is such that crystalline MgO (periclase) cannot form. Petrographic studies have failed to reveal the presence of periclase in high-MgO-content slags proposed for use in making portland blast-furnace slag cement. However, it is stated that studies by McCaffery at the University of Wisconsin in 1927 revealed periclase in at least one slag examined at that time. In blast-furnace operation, effective, practically useful slags combine three characteristics: they are completely liquid at furnace temperatures; they have maximum ability to extract sulfur from the rest of the blast-furnace charge; and they have minimum viscosity. If these conditions are achieved, the slag compositions do not fall in a range where periclase can be produced if the liquid is rapidly quenched, as it will be in effective granulation.[8] Particular attention was paid to the possibility of the occurrence of crystalline MgO (periclase) in the slags, especially in Slag 336 which was found to contain 9.65 percent MgO by chemical analysis. According to Bogue[9], crystalline MgO can be resolved by x-ray diffraction when present in portland cement in amounts of 2.5 percent or more. This degree of resolution was reported in work by Bogue and Brownmiller in which equipment of very much lower resolving power was employed than that used in the present study. Although work on synthetic mixtures to determine the resolving power of our equipment has not been completed, it is believed that crystalline MgO would be detected if present in amounts of 0.5 percent or more. The x-ray diffraction records on all samples of slag, including Slag 336, did not show any indication of crystalline MgO. It is therefore regarded as certain that periclase, if present, amounts to less than 2.5 percent and probably less than 0.5 percent.

Data on the crystalline constituents are given in Table 2; all except metallic iron were detected in the x-ray diffraction patterns. All samples of slag containing calcite revealed the presence of this constituent by effervescence when a sample being examined under the microscope was treated with dilute hydrochloric acid. The presence of metallic iron was established by behavior of particles in a magnetic field. The metallic iron in 340 was also detected by specific gravity. Examination of immersion mounts with a polarizing microscope permitted identification, by optical properties, of quartz in five of the seven slags containing it, and of melilite in two of six

slags containing melilite. The amount of crystalline material in the slags is small, as it should be when they are used as hydraulic materials. Crystalline material is well dispersed through the glassy matrix and the crystals other than calcite and quartz are in most cases completely surrounded by glass. Crystalline material probably does not make up as much as 10 percent and certainly not more than 15 percent of any of the slags. Slag 339 has the least amount of crystalline material. It is suspected that the quartz found as a minor constituent of all the slags except 340 is a contaminant introduced during or after granulation. The failure to identify calcium sulfide and manganese sulfide may be more related to the inaccuracy of the available x-ray data and the interference of other constituents, particularly melilite, than with the actual presence or absence of the sulfides. However, it is reasonable to expect the crystalline sulfides to be absent from these slags in view of their relatively low sulfide content and very high degree of glassiness.

The work of Osborn and associates[8] has frequently been referred to in connection with the discussion of the crystalline phases that may be present in blast-furnace slags and hence in portland blast-furnace slag cement. The diagrams for CaO-MgO-SiO_2-Al_2O_3 presented by Osborn, particularly those for CaO-MgO-SiO_2 at 10 percent and 15 percent Al_2O_3, have been used to plot the points for the eight slags studied in this investigation with the following comparative results:

Slag No.	Composition calculated to 100 percent from chemical analysis				Primary phase field for plotted point	Crystalline constituents found by x-ray diffraction
	Al_2O_3	SiO_2	CaO	MgO		
336	11.49	35.04	42.97	10.50	Merwinite*	Merwinite
337	10.04	36.76	47.77	5.43	C_2S*	$\alpha'C_2S$, melilite
340	11.06	37.10	47.78	4.06	C_2S*	Melilite, C_2S, αCS
345	11.66	38.88	43.69	5.77	Melilite*	Melilite, C_2S
338	12.89	34.35	46.50	6.26	C_2S—Melilite†	Melilite, C_2S
339	12.08	38.45	45.96	3.51	Melilite†	None
341	13.51	36.72	47.24	2.23	Melilite†	Melilite, C_2S
342	12.72	34.75	49.02	3.51	Melilite†	Melilite, αCS, C_2S

*Using 10 percent Al_2O_3 diagram.
†Using 15 percent Al_2O_3 diagram.

Samples of each slag were examined in immersion media using the polarizing microscope to determine the index of refraction of the glass (Table 2). Studies by George and others[10] have indicated an inverse relationship between silica content and index of refraction of natural volcanic glass. Previous studies (unpublished) have indicated this relationship to be apparently applicable to the glass content of iron blast-furnace slags and also to the glass in fly ash (Fig. 1). Points marked "198" and "216" refer to the samples of blast-furnace slag used in the lean mortar-bar tests. Particle counts were made of two samples of each slag, identifying clear particles as "glassy,"* mixed particles, and opaque particles. Percentage of glass was estimated by adding percentage of clear particles to one-half the percentage of mixed par-

*This method is modified from one described by Parker and Nurse.[11]

TABLE 2—PETROGRAPHIC AND X-RAY DIFFRACTION DATA ON SLAGS USED IN MAKING PORTLAND BLAST-FURNACE SLAG CEMENTS*

Data	Serial No.							
	336	345	338	341	339	340	337	342
SiO_2, percent (chemical analysis)	32.21	37.80	32.11	33.58	37.25	35.92	35.72	33.39
Index of refraction of glass, lower and higher limits	1.470 1.610	1.640 1.650	1.636 1.660	1.610 1.650	1.640 1.660	1.640 1.660	1.620 1.654	1.640 1.660
Crystalline components								
Calcite ($CaCO_3$)	Major	†	Major	Major	†	Minor	†	Minor
Merwinite ($3CaO \cdot MgO \cdot 2SiO_2$)	Minor	†	†	†	†	†	†	†
Quartz (SiO_2)	Minor	Minor	Minor	Minor	Minor	†	Minor	Minor
Bredigite ($\alpha' C_2S$)	†	†	†	†	†	†	Major	†
Melilite ($nC_2AS \cdot mCaMS_2$)	†	Major	Minor	Minor	†	Major	Minor	Major
Iron (Fe)	†	Minor	Minor	Minor	Minor	Minor	Minor	Minor
Beta C_2S	†	†	Minor	†	†	†	†	†
Pseudowollastonite (αCS)	†	†	†	†	†	Minor	†	Minor
C_2S (form unknown)	†	Minor	†	Minor	†	Minor	†	Minor
Approximate glass content, percent	60	80	60-80	60	90	60-80	80	60-80

*Arranged in order of increasing CaO content by chemical analysis.
†Not detected.

ticles (Table 2). It is believed that the glass content so indicated is lower than is correct and that, as stated above, crystalline materials do not make up as much as 10 percent and certainly not more than 15 percent of any of the slags.

PORTLAND CEMENT CLINKERS

Table 3 indicates that the eight portland cement clinkers all comply with the requirements of federal specifications SS-C-197; however, none would be classed as Type II since all but one contain more than 8 percent C_3A and the one containing less than 8 percent C_3A contains more than 58 percent $C_3A + C_3S$.

TABLE 3—CHEMICAL DATA ON PORTLAND CEMENT CLINKERS USED IN MANUFACTURE OF PORTLAND BLAST-FURNACE SLAG CEMENTS

Constituents, percent	Specification limits	Serial No.							
		337	338	339	340	341	345	342	336
Al_2O_3	Max. 7.5*	6.01	6.43	5.69	7.06	6.04	6.45	6.25	2.87
Fe_2O_3	Max. 6.0*	3.17	3.24	3.77	2.88	2.11	2.85	3.56	3.20
MgO	Max. 5.0*	2.96	2.44	3.37	2.50	2.09	2.23	1.19	1.01
C_3A	Max. 15*	10.6	11.6	8.7	13.8	12.4	12.3	10.5	2.2
SiO_2	Min. 21.0†	21.28	21.27	21.27	21.49	23.85	21.02	21.61	24.80
$C_3S + C_2A$	Max. 58†	65.3	66.8	64.1	56.4	53.1	67.6	61.8	66.1
Insoluble residue	Max. 0.75†	0.20	0.08	0.08	0.04	0.40	0.19	0.10	0.07
Loss on ignition	Max. 3.0†	0.91	0.18	1.34	1.80	0.24	0.94	1.79	0.35
Moisture loss	—	0.15	0.05	0.08	0.14	0.08	0.18	0.14	0.05
CaO	—	64.41	65.42	64.15	63.43	65.26	64.88	64.64	67.89
Mn_2O_3	—	0.69	0.47	0.27	0.04	0.02	0.04	0.14	0.05
SO_3	—	0.28	0.54	0.15	0.23	0.02	0.55	0.15	0.04
S	—	0.07	0.02	0.00	0.01	0.00	0.01	0.00	0.00
Na_2O	—	0.13	0.11	0.09	0.17	0.14	0.28	0.09	0.15
K_2O	—	0.11	0.25	0.08	0.62	0.59	0.78	0.47	0.09
Total as Na_2O	—	0.20	0.27	0.14	0.58	0.53	0.79	0.40	0.21
C_3S	—	54.7	55.2	55.4	42.6	40.7	55.3	51.3	63.9
C_2S	—	19.8	19.4	19.3	29.5	37.8	18.6	23.3	23.0
C_4AF	—	9.6	9.8	11.5	8.8	6.4	8.7	10.8	9.7

*Federal specification SS-C-197.
†Federal specification SS-C-192b, for Type II portland cement.

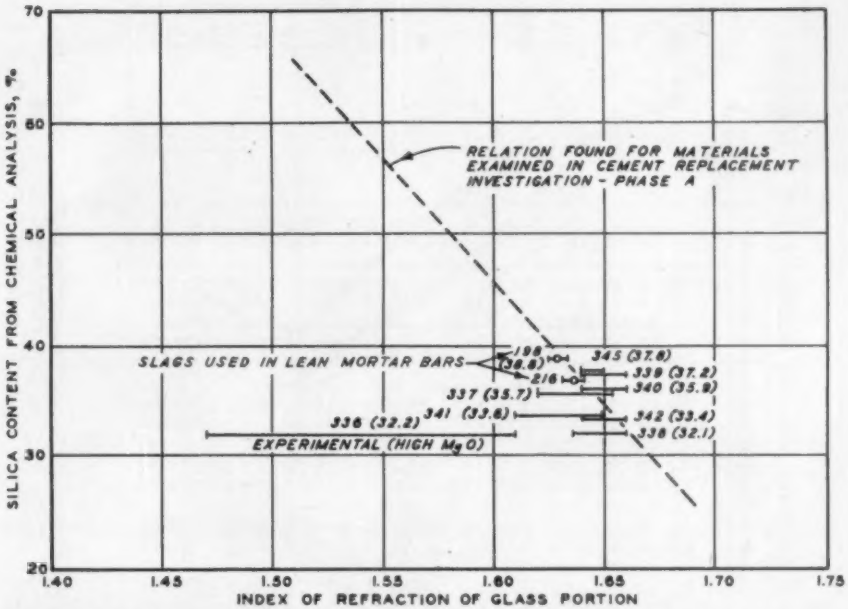

Fig. 1—Relation between index of refraction of glass and silica content of blast-furnace slags

The portland cement clinkers were examined by x-ray diffraction and the presence of crystalline MgO was detected in all of them. A measure of counts per second above background at the 2.106-A spacing was made and the cements were ranked in order of crystalline MgO content on the basis of these counts. These results compare with MgO content calculated from chemical analysis as indicated below:

Slag No.	MgO by chemical analysis		Periclase by x-ray diffraction, rank
	Percent	Rank	
339	3.37	1	1
337	2.96	2	2
340	2.50	3	3 or 4
338	2.44	4	3 or 4
345	2.23	5	5 or 6
341	2.09	6	5 or 6
342	1.19	7	7
336	1.01	8	8

PORTLAND BLAST-FURNACE SLAG CEMENTS

Results of tests on the portland blast-furnace slag cements and the reference Type II portland cement are given in Table 4. The rate of compressive strength development is given in Fig. 2. Several of the portland blast-furnace

TABLE 4—RESULTS OF TESTS OF PORTLAND BLAST-FURNACE SLAG CEMENTS AND THE REFERENCE TYPE II PORTLAND

Determination	Specification requirements		Commercial portland blast-furnace slag cements							Non-commercial (experimental high-MgO slag)	Reference Type II
			Type IS						Type IS-A		
	Federal SS-C-197	ASTM C 205-53T	337	338	339	340	341	345	342	336	330
Specific gravity	—	—	3.03	3.03	3.02	3.04	3.00	3.03	3.03	2.99	3.17
Specific surface, air permeability, sq cm per g, average	Minimum 3400	Minimum 3400	4175	4745	4120	3685	3995	4160	4565	6085	3590
Passing No. 325 sieve, percent	Minimum 88	Minimum 88	95.2	95.4	97.0	90.8	84.6*	93.1	95.1	96.9	94.8
Normal consistency, percent	—	—	28.0	27.0	32.0	28.2	24.2	29.0	28.6	26.8	24.6
Autoclave expansion, percent	Maximum 0.20	Maximum 0.20	−0.01	0.02	0.03	0.02	0.05	0.04	−0.02	0.05	0.10
Compressive strength, psi; 3 days	Minimum 900†	Minimum 900†	1540	2400	1765	1790	1690	1880	—	2365	1745
	750‡	750‡	—	—	—	—	—	—	1600	—	—
7 days	Minimum 1800†	Minimum 1800†	2490	4000	2915	2940	3100	3050	—	4010	2665
	1500‡	1500‡	—	—	—	—	—	—	2475	—	—
28 days	Minimum 3000	Minimum 3000	4965	6115	5960	5050	4825	5340	3875	5325	5140
Air content, mortar, percent	Maximum 12†	—	4.4	4.6	5.9	4.6	3.8	4.9	—	6.7	8.8
	16 ± 4‡	Maximum 16 ± 4‡	—	—	—	—	—	—	15.9	—	—
Setting time, hr: min, Vicat	—	Minimum 0:45									
		Maximum 7:00	2:05	1:55	3:30	3:15	1:50	2:50	2:45	0:05*	2:55
Gillmore, initial	Minimum 0:60	—	4:45	4:15	4:30	3:40	2:55	4:15	3:10	0:45*	5:30
final	Maximum 10:00	—	9:15	7:30	7:30	6:40	5:45	7:0	6:25	9:0	8:0
Slag modulus: $\frac{CaO + MgO + 1/3\ Al_2O_3}{SiO_2 + 2/3 + Al_2O_3}$	Minimum 1	Minimum 1	1.30	1.33	1.15	1.25	1.19	1.14	1.30	1.34	—
MgO, percent	Maximum 5.0	Maximum 5.0	3.66	4.11	3.20	2.91	1.90	3.65	2.25	7.57*	2.8
SO_3, percent	Maximum 2.5	Maximum 2.5	2.22	2.39	2.12	2.69*	1.28	2.37	2.59*	2.52*	2.0
Mn_2O_3, percent	Maximum 1.5	Maximum 1.5	0.97	0.93	0.38	0.38	0.12	0.27	0.49	0.08	—
S, percent	Maximum 2.0	Maximum 2.0	0.42	0.73	0.67	0.36	0.28	0.28	0.47	0.63	—
Insoluble residue, percent	Maximum 1.0	Maximum 1.0	0.50	0.86	0.40	0.54	1.08*	0.56	0.49	0.39	0.23
Loss on ignition, percent§	Maximum 3.0	Maximum 3.0	0.76	0.81	−0.28	1.08	2.80	0.60	0.64	0.19	1.6
Moisture, percent	—	—	0.35	0.38	0.19	0.32	0.25	0.07	0.19	0.21	—
SiO_2, percent	—	—	26.43	24.15	26.46	24.46	24.89	26.82	25.13	28.60	23.10
Al_2O_3	—	—	7.06	9.16	8.63	7.45	6.78	7.24	8.46	6.10	5.00
Fe_2O_3, percent	—	—	2.30	2.40	2.19	2.27	1.63	2.00	2.53	1.63	3.9
CaO, percent	—	—	54.85	52.78	55.05	56.63	58.70	55.31	56.22	49.71	61.1
Na_2O, percent	—	—	0.14	0.16	0.13	0.17	0.13	0.30	0.10	0.33	0.23
K_2O, percent	—	—	0.16	0.34	0.12	0.76	0.69	0.69	0.48	0.48	0.69
Total as Na_2O, percent	—	—	0.25	0.38	0.21	0.67	0.58	0.75	0.41	0.65	0.68

*Noncompliance with federal or ASTM specifications.
†Requirement for Type IS.
‡Requirement for Type IS-A.
§A negative value indicates a gain in weight on ignition; such gain is believed due to oxidation of sulfide sulfur and iron or manganese constituents.

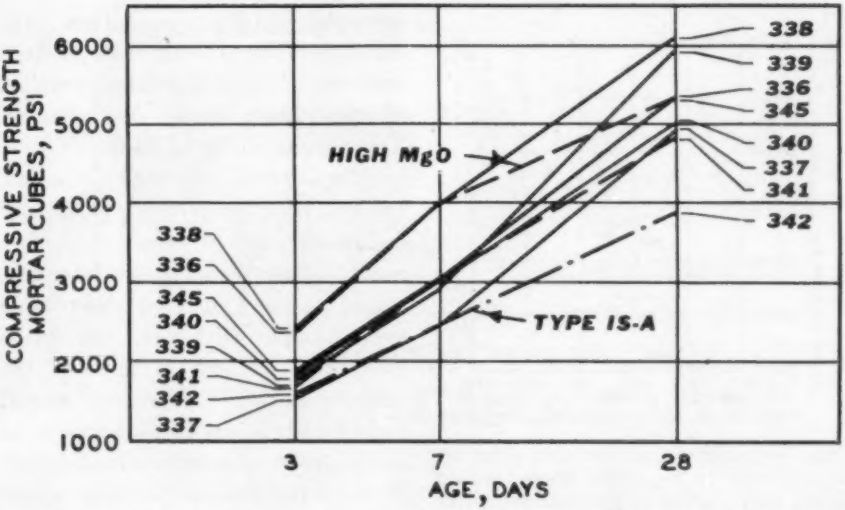

Fig. 2—Rate of development of compressive strength of portland blast-furnace slag cements in mortar

slag cements did not meet one or more of the requirements of federal or ASTM specifications. Of the commercial cements, 341 showed excessive insoluble residue and deficient amount passing No. 325 sieve; Cements 340 and 342 had excessive SO_3.

Slag content

Federal specifications SS-C-197 require (Table 14) that the slag constituent shall be not less than 25 percent and shall not exceed 65 percent of the portland blast-furnace slag cement by weight. No means were found in the course of these tests by which a sample of portland blast-furnace slag cement could be tested in the laboratory in order accurately to determine the percentage by weight of the slag constituent. Data on the percentage of slag used in the manufacture of the cements studied, as provided by the producers, are given in Table 1. It will be noted that the entire permissible range is represented, Cement 341 being stated to contain 25 percent slag and Cement 336, 62 percent. Fig. 3 shows that the portland blast-furnace slag cements containing the larger percentages of slag tend to be more finely ground.

X-ray diffraction results

Routine x-ray diffraction examination is sufficient to distinguish a portland cement clinker from its companion portland blast-furnace slag cement on the basis that the maximum peak height is always greater on the x-ray pattern of the clinker (Fig. 4). There is a rough correlation between maximum peak height on an x-ray pattern and the slag content of a portland blast-furnace slag cement. The peak height decreases as the slag content

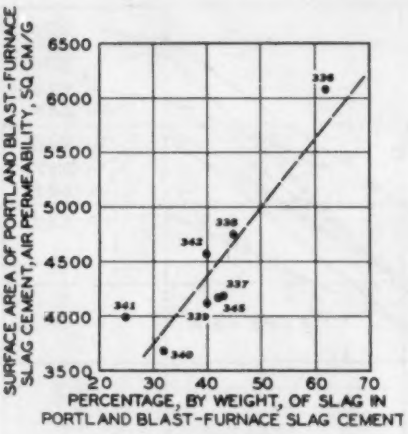

Fig. 3—Portland blast-furnace slag cements tend to be more finely ground when they contain larger percentages of slag

increases. At the present time x-ray diffraction will not enable all portland cements to be distinguished from all portland blast-furnace slag cements. The reproducibility of maximum peak heights in terms of counts per second has not been completely satisfactory apparently due to excessive particle size. Samples tested to date have contained particles up to 44 microns in size. Samples with x-ray absorption coefficients as high as those of the calcium silicates in cement should have a particle size below 1 micron to yield consistent results. Peak-height data for the eight cements are given below:

MAXIMUM PEAK HEIGHTS, BACKGROUND LEVELS, AND NET PEAK HEIGHTS, COUNTS/SEC

Serial No.	Portland cement clinker			Portland blast-furnace slag cement			Slag content of cement, percent
	Peak*	Background†	Net peak height	Peak*	Background†	Net peak height	
341	860	160	700	710	160	550	25
340	835	170	665	725	150	575	32
339	950	210	740	660	200	460	40
342	780	200	580	590	180	410	40
345	890	220	670	590	170	420	42
337	930	180	750	545	160	385	43
338	860	170	690	635	230	405	45
336	1110	150	960	560	160	400	62

*Manually scaled on a combined CaS-CaS peak; averages of nine packings per specimen.
†From charts; projected background under the peak scaled.

Sulfate content, false set

The portland blast-furnace slag cements and the reference portland cement were tested for the presence of calcium sulfate in hydrated mortar at an age of 24 hr using the procedures described in ASTM C 265-55T except that the mortar samples were placed in plastic bags and stored in a controlled-temperature water bath. The portland blast-furnace slag cements were tested for penetration (false set) according to ASTM C 359-55T. The results of these tests are given in Table 5.

Effect of reduced temperature on setting time

The effect of reduced temperature on setting time of portland blast-furnace slag cements and the effectiveness of the use of 2 percent $CaCl_2$ in correcting retardation caused by reduced temperature were determined. Table 6 shows Vicat setting-time test results for four conditions: (a) no $CaCl_2$, 73.4 ± 2 F; (b) no $CaCl_2$, 40 ± 5 F; (c) 2 percent $CaCl_2$, 73.4 ± 2 F; and

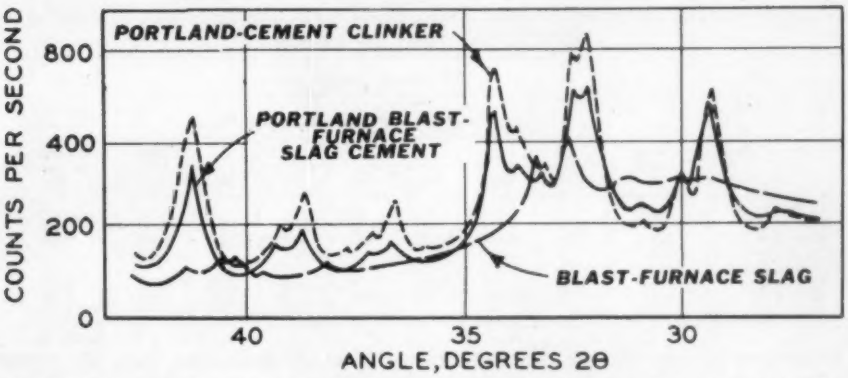

Fig. 4—X-ray spectrometer patterns for Cement 337 (43 percent slag) and for the clinker and slag used in its manufacture

(d) 2 percent $CaCl_2$, 40 ± 5 F. All of the commercial portland blast-furnace slag cements comply with the specified requirements on setting time (Table 4) when tested at 73.4 ± 2 F. When tested at 40 ± 5 F, four of them show excessive retardation; adding 2 percent $CaCl_2$ brought the setting time back within the specified limits in these cases. Addition of 2 percent $CaCl_2$ at 73.4 ± 2 F caused one of the cements that was not excessively retarded at 40 F to be excessively accelerated.

Heat of hydration

Previous studies [12,13] have shown that experimentally determined heats of hydration of blends consisting of known proportions of portland cement and another material having a known but lower heat of hydration cannot be predicted by calculation. The determined heat of hydration will be higher than such calculations indicate. When the other material is used in only moderate amounts or has a heat of hydration close to that of the portland cement with which it is mixed, the heat of hydration of the blend will in some cases be higher than that of the portland cement alone. Some of these indications may be more apparent than real, since unless the difference is more than 5 calories per g the difference in results may represent merely

TABLE 5—RESULTS OF TESTS FOR CALCIUM SULFATE IN HYDRATED MORTARS AND PENETRATION OF MORTARS (FALSE SET)

Cement		SO_3, 24 hr, g per liter	Water added to mortar, ml	Temperature, F	Depth of penetration, mm				
No.	Type				Initial	5 min	8 min	11 min	Remix
330	II	0.00	180	73	35	3½	3¼	2¼	47*
337	IS	0.15	220†	72	50+	0	0	0	50+
338	IS	0.00	220†	73	50+	50+	31	35	50+
339	IS	0.00	220†	73.5	50+	1	0	0	50+
340	IS	0.05	220†	74	49	2	½	½	50+
341	IS	0.00	220†	74.5	50+	50+	50+	50+	50+
345	IS	0.00	220†	75	50+	1	1	½	50+
342	IS-A	0.00	220†	74	50+	0	0	0	50+
336	IS	1.18	220†	73	50+	2	½	0	50+

*Average of two determinations.
†Amount of water that was required to give initial penetration of about 50 mm.

TABLE 6—EFFECT OF TEMPERATURE AND USE OF CALCIUM CHLORIDE ON SETTING TIME (VICAT TEST) OF CEMENTS

Test conditions	Setting time, hr: min								
	Type II portland cement	Commercial portland blast-furnace slag cement							Experimental high-MgO portland blast-furnace slag cement
		Type IS						Type IS-A	
	330	337	338	339	340	341	345	342	336
No $CaCl_2$:									
73.4 ± 2 F	2:55	2:05	1:55	3:30	3:15	1:50	2:50	2:45	0:05
40 ± 5 F	5:35	8:00	4:55	11:00	9:15	5:00	7:55	6:40	0:05
2 percent $CaCl_2$:									
73.4 ± 2 F	0:50	1:30	1:25	1:00	1:30	0:35	1:10	0:50	0:08
40 ± 5 F	1:45	3:15	2:20	2:15	2:55	1:20	2:15	1:20	0:10

experimental error in the determination. Heat of hydration data are given in Table 7.

Only Cements 338 at 7 days age and Cement 339 at 7 and 28 days age show heats of hydration greater than permitted under the optional performance requirement of the federal specification for Type II portland cement. It is believed that most portland blast-furnace slag cements manufactured to comply with present specifications for Type IS and IS-A will meet the optional requirements of Type II portland for heat of hydration; it is also believed that all portland blast-furnace slag cements would meet the requirements if Type II portland clinkers were used in their manufacture.

Using the adiabatic temperature rise, specific heat, and mixture proportion data for the 6-in. aggregate concrete mixtures, values were computed for heat of hydration at 3, 7, and 28 days age. These data are also given in Table 7.

Sulfate resistance

The nine cements were tested for sulfate resistance using mortar bars containing added calcium sulfate in amounts sufficient to bring the SO_3 content to 7.0 percent by weight of the cement.[14] The results of these tests (Table 8) suggest that sulfate resistance varies inversely with the C_3A content of the portland cement clinker component. All but one of these clinkers contained more than 8 percent calculated C_3A. All the portland blast-

TABLE 7—HEAT OF HYDRATION DATA

Serial No.	Conduction calorimeter method*			Heat of solution method*			Computed from adiabatic temperature rise of concrete		
	1 day	3 days	7 days	7 days	28 days	1 yr	3 days	7 days	28 days
330	35	54	67	71	82	89	28	43	45
337	26	45	60	68	77	88	33	39	48
338	36	54	68	75	80	91	38	52	57
339	33	52	67	79	82	96	34	47	57
340	38	57	69	70	80	90	35	49	58
341	30	47	66	64	76	84	33	49	56
345	41	55	66	62	73	81	29	34	36
342	37	55	67	64	74	84	43	55	58
336	24	48	62	64	68	78	27	40	45
Specifications for Type II, maximum	—	—	—	70	80	—	—	—	—

*Methods are described in Reference 13.

TABLE 8—SULFATE RESISTANCE OF PORTLAND BLAST-FURNACE SLAG CEMENTS AND THE REFERENCE PORTLAND CEMENT

Serial No.	Type	Portland cement clinker C_3A, percent	Clinker, percent	Percent expansion at age, days				
				7	28	84	140	365
330	II Portland	7.0	100	0.036	0.065	0.100	0.115	0.136
337	IS	10.6	53	0.051	0.100	0.159	0.155	0.162
338	IS	11.6	51	0.097	0.159	0.193	0.164	0.172
339	IS	8.7	(60)*	0.057	0.154	0.426	0.428	0.436
340	IS	13.8	62	0.056	0.144	0.451	0.690	0.690
341	IS	12.4	(75)*	0.049	0.104†	0.188†	0.282†	0.392†
345	IS	12.3	53	0.045	0.100	0.252	0.292	0.295
342	IS-A	10.5	60	0.049	0.105	0.202	0.200	0.203
336	IS	2.2	34	0.036	0.055	0.070	0.074	0.089

*Value based on reported slag content.
†Later tests suggest that these values are incorrect and that expansion at 365 days for cement 341 should be approximately 0.75 percent.

furnace slag cements, except the one made with low C_3A portland cement clinker showed more than 0.10 percent expansion at 84 days, the approximate average expansion of portland cements of 8 percent C_3A content.[14] Cement 340, containing 13.8 percent C_3A in its clinker constituent, showed maximum expansion.

Since C_3A content of the portland cement clinker constituent appeared to be closely related to sulfate resistance of portland blast-furnace slag cements, consideration was given to other means than theoretical calculation from chemical analysis for estimating relative C_3A content. The portland cement clinker samples were examined by x-ray diffraction and a measure of counts per second above background at the 2.70-A spacing was taken as an index of C_3A content, since this spacing is characteristic of C_3A and is less subject to interference by other constituents of portland cement than other C_3A spacings. All the clinkers except 336 showed a peak at this spacing. Clinker 336 may therefore be regarded as having least C_3A and the amount contained may be regarded as below the limit of resolution by the techniques used, in the presence of the other constituents of the clinker.

Arranging the portland blast-furnace slag cements in order of expansion of sulfate resistance test specimens at 365 days, the relationships of relative C_3A content by x-ray diffraction and calculation from chemical analysis are as shown in Table 9.

Tuthill[15] suggests that the presence of active silica or pozzolan is helpful in reducing the severity of sulfate attack, since these constituents combine with calcium hydroxide, making much of it insoluble, and thus avoiding the condition of concentrated calcium hydroxide solutions in the concrete, which has been shown to be the condition under which calcium sulfoaluminate forms most rapidly. Blast-furnace slag has been spoken of favorably in European literature for use in concrete for sea-water exposure. The failure of these portland blast-furnace slag cements to show better sulfate resistance in mortar tests than would have been expected from tests of their portland-cement component alone suggests either that: (a) the test permitted sulfate reaction at such an early age that the activity of the silica in the slag had not

TABLE 9—COMPARISON CALCULATED OF C_3A CONTENTS

Serial No.	Percent expansion of sulfate resistance test specimens at 365 days	Clinker in portland blast-furnace slag cement, percent	C_3A content: Clinker: Counts per second, 2.70-A peak*	Clinker: Calculated from chemical analysis: Standard method,† percent	Clinker: Calculated from chemical analysis: Swayze method,‡ percent	Portland blast-furnace slag cement: Counts per second × 10^{-4} × percent of clinker	Portland blast-furnace slag cement: Percent clinker × calculation from chemical analysis: Standard method, percent	Portland blast-furnace slag cement: Percent clinker × calculation from chemical analysis: Swayze method, percent
340	0.690 (1)§	68	215 (1)§	13.8 (1)§	12.1 (1)§	14.6 (1)§	9.4 (1)§	8.2 (1)§
339	0.436 (2)	60	130 (5.6)	8.7 (7)	8.7 (4)	7.8 (5)	5.2 (7)	5.2 (4)
341	0.392 (3)††	75	190 (3)	12.4 (2)	8.9 (3)	14.2 (2)	9.3 (2)	6.7 (2)
345	0.295 (4)	58	130 (5.6)	12.3 (3)	7.4 (5)	7.5 (6)	7.1 (3)	4.3 (5)
342	0.203 (5)	60	200 (2)	10.5 (6)	4.5 (7)	12.0 (3)	6.3 (5)	2.7 (7)
338	0.172 (6)	55	150 (4)	11.6 (4)	10.9 (2)	8.2 (4)	6.4 (4)	6.0 (3)
337	0.162 (7)	57	125 (7)	10.6 (5)	5.2 (6)	7.1 (7)	6.0 (6)	3.0 (6)
336	0.089 (8)	38	1** (8)	2.2 (8)	2.2 (8)	0.4 (8)	0.8 (8)	0.8 (8)

*Height of 2.70-A peak was scaled three times at each of three locations on sample surface. Background was calculated from a straight line drawn between 20 and 40 deg 2 Θ and subtracted from mean of nine scalings to give value tabulated.
†As given in Note 2 to Table 1 of federal specification SS-C-192a.
‡As given in *American Journal of Science*, V. 244, 1946: If Al_2O_3/Fe_2O_3 ratio is 1.6 or greater and the clinker contains sufficient MgO or alkalies to lower the invariant temperature to 1300 C or lower, the ferrite phase will be C_4AF and 1.276 × weight of Fe_2O_3 will give weight of Al_2O_3 in C_4AF. Total Al_2O_3 − Al_2O_3 in C_4AF = residual Al_2O_3. Residual Al_2O_3 × 2.65 = C_3A. Otherwise calculate as in † above.
§Values in parentheses are relative rank from 1 to 8.
**No peak was found; a count of one was assumed in order to provide a basis for calculation.
††See note to Table 8.

become effective in reducing calcium hydroxide concentration; or (b) the slags contained nearly sufficient calcium to combine with their silica and hence the silica did not deplete the calcium hydroxide concentration when it did become active. In any case the mortar test with excess calcium sulfate may be regarded as providing a basis for separating portland blast-furnace slag cements made with high-C_3A portland clinkers from those made with low-C_3A portland clinkers.

In an attempt to resolve this question, additional tests were undertaken using two high-C_3A portland cements and two granulated blast-furnace slags. Lean-mortar bars, without added calcium sulfate, were made and tested[16] using each of the cements and with 40 percent of each cement (by solid volume) replaced by each of the two slags. The bars without slag showed expansions of more than 0.10 percent after 34 to 37 days storage in a 5 percent solution of sodium sulfate; the bars containing slag showed average expansions of from 0.034 to 0.046 percent after 77 days (Fig. 5). Bars with slag and Cement 332 showed expansion of more than 0.1 percent at 154 and 196 days and were discontinued; bars with slag and Cement 334 showed less than 0.06 percent expansion at 226 days. It is suggested that the test using added calcium sulfate does not allow the slag constituent of portland blast-furnace slag cements to manifest its effect in promoting sulfate resistance.

Alkali-aggregate reaction

The federal specification for portland cements includes a limit of 0.60 percent on alkalies (Na_2O + 0.658 percent K_2O) that is effective when the purchaser specifies "low-alkali cement," as would be appropriate when the cement is to be used with "reactive" aggregate. Barona[17] and others have reported that portland blast-furnace slag cements containing 50 to 60 per-

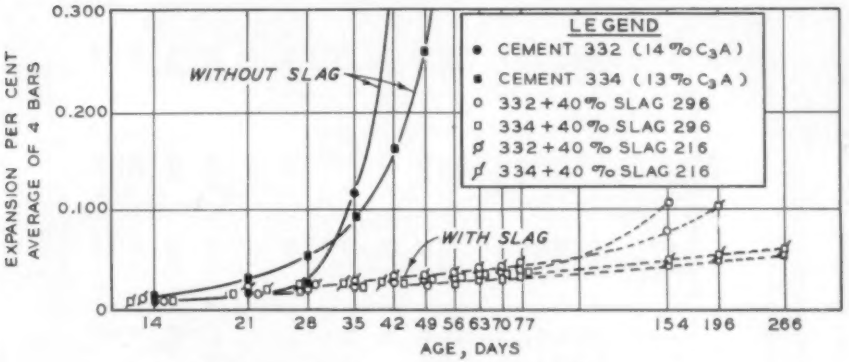

Fig. 5—Expansion of lean mortar bars in sulfate solution

cent slag of the type contemplated by ASTM specifications for portland blast-furnace slag cement will serve to prevent excessive expansion of mortar bars and concrete made with reactive aggregate. In Barona's tests with pyrex glass aggregate, NaOH was added to the mixing water to give 1.23 percent Na_2O equivalent in all mortar bars. The slag-portland blends showed greatly reduced expansion as compared with the mortars containing no slag.

In these tests mortar bars were made using pyrex glass aggregate and each cement. Mortars were proportioned to a flow of 110 ± 5 percent; otherwise the procedures were as given in CRD-C 123.[6] At 168 days age the expansion of bars with Type II portland was 1.104 percent; that of all portland blast-furnace slag cement mortars was less than 0.050; that of six cements, 336, 337, 338, 339, 341, and 342, was zero or less (Table 10). Bars made in another program with the same aggregate and flow using a very high-alkali cement (211, 1.07 percent Na_2O equivalent) show expansions of 0.068 percent at 8 days, 0.139 percent at 28 days, and 0.201 percent at 84 days. Results of chemical analysis of cements, clinkers, and slags in this program for Na_2O equivalent are given in Table 10. It would appear that the presence of slag in the portland blast-furnace slag cements acts to keep expansion from becoming excessive even when a highly reactive aggregate is used.

TABLE 10—RESULTS OF MORTAR-BAR TEST FOR REDUCTION IN EXPANSION DUE TO ALKALI-AGGREGATE REACTION

Serial No.	Na_2O equivalent, percent			Percent length change at age, days				
	Cement	Clinker	Slag	8	28	84	168	365
330	0.68	—	—	+0.049	+0.084	+0.098	+0.104	+0.123
336	0.65	0.21	1.08	+0.007	+0.001	0.000	−0.005	+0.003
337	0.25	0.20	0.37	+0.002	−0.008	−0.011	−0.013	−0.003
338	0.38	0.27	0.51	−0.001	−0.007	−0.013	−0.014	−0.006
339	0.21	0.14	0.47	+0.004	−0.008	−0.014	−0.017	−0.007
340	0.67	0.58	0.48	+0.023	+0.024	+0.018	+0.016	+0.026
341	0.58	0.53	0.28	+0.001	−0.008	−0.011	−0.013	−0.005
342	0.41	0.40	0.47	+0.002	−0.007	−0.011	−0.016	−0.007
345	0.75	0.79	0.74	+0.036	+0.040	+0.043	+0.041	+0.053
211	1.07	—	—	+0.068	+0.139	+0.201	—	—

TABLE 11—RESULTS OF TESTS OF ¾-IN.-AGGREGATE CONCRETE MADE WITH PORTLAND CEMENT AND PORTLAND BLAST-FURNACE SLAG CEMENTS

Test data	Type II portland cement	Commercial portland blast-furnace slag cement							Experimental high-MgO portland blast-furnace slag cement
		Type IS						Type IS-A	
	330	337	338	339	340	341	345	342	336
Bleeding, percent	3.0	4.1	3.1	3.5	5.2	3.8	4.1	2.1	1.9
Compressive strength,* psi, 3 days	2210	1300	1910	1400	1620	1450	1500	1510	1810
7 days	2960	2350	3050	2250	2320	2600	2290	2330	2970
28 days	5020	4100	4360	3710	3550	4120	3390	3580	3790
90 days	6180	4940	5070	4310	4230	4690	4230	4290	4570
180 days	6410	5280	5440	4810	4610	4970	4460	4590	4730
365 days	6760	5660	6020	5360	5110	5730	4820	5030	5230
Flexural strength,† psi, 7 days	675	600	705	585	655	645	635	635	780
28 days	880	910	955	845	845	830	840	855	955
90 days	905	980	985	920	830	790	845	860	910
Resistance to freezing and thawing, DFE_{300}	51	45	44	53	41	52	34	52	41
Permeability, $K_c \times 10^6$ 90 days	7	7	6	16	6	13	5	8	6
180 days	4	3	3	5	3	4	3	3	4
Shrinkage,‡ 180 days, percent $\times 10^3$	49	48	52	56	48	47	43	51	44
Expansion, 180 days, percent $\times 10^3$	6	3	−1	1	0	−2	4	−1	2
Young's Modulus, 28 days, psi $\times 10^{-6}$	5.01	4.87	4.80	4.52	4.98	4.85	4.89	4.53	4.82
Abrasion loss, g per sq ft	104	124	101	117	122	117	131	133	115
Subsidence,§ percent $\times 10^2$, 1 day	12	17	18	19	9	5	6	22	15
3 days	11	19	12	19	7	6	7	26	15
7 days	11	18	15	22	7	4	8	28	18

*Each value represents average of nine tests of 3- x 6-in. cylinders.
†Each value represents average of nine center-point load tests of 3½- x 4½- x 16-in. beams.
‡Using Corps of Engineers Method CRD-C 56, calculating shrinkage from length at 14 days age.
§Using procedure described in text, calculating subsidence from initial reading taken before final setting.

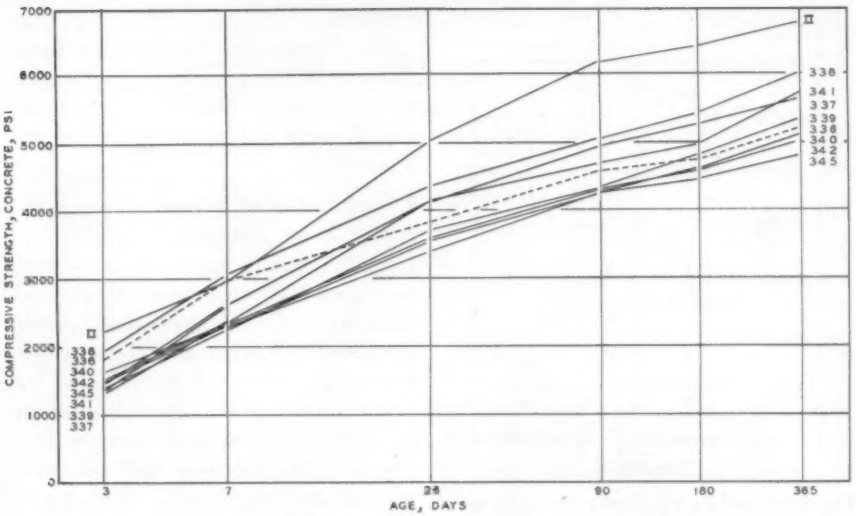

Fig. 6—Rate of development of compressive strength of portland blast-furnace slag cements and a Type II portland in concrete

SMALL-AGGREGATE CONCRETE MIXTURES

Mixture data

Nine concrete mixtures were proportioned and tested, six made with the commercial Type IS portland blast-furnace slag cements, and one each made with Type II portland, Type IS-A portland blast-furnace slag, and experimental high-MgO portland blast-furnace slag cement. Aggregates were a natural siliceous sand from Mississippi graded to a fineness modulus of 2.65 ± 0.04 and a crushed limestone from Tennessee graded to ¾-in. size. The actual cement factor of all the mixtures was in the range 5.46 to 5.49 bags per cu yd; the water-cement ratio was 5¼ ± 0.2 gal. per bag; air content was in the range of 6.0 ± 0.3 percent; and the slump from 2 to 2¾ in. Test data are given in Table 11. Rates of development of compressive and flexural strength are indicated in Fig. 6 and 7.

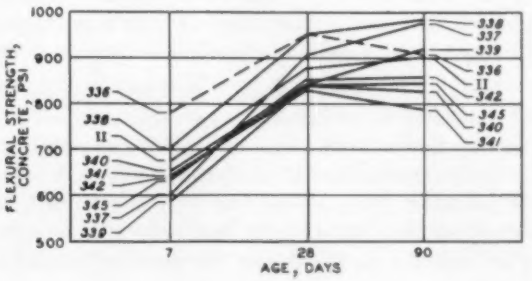

Fig. 7—Rate of development of flexural strength of portland blast-furnace slag cements and a Type II portland in concrete

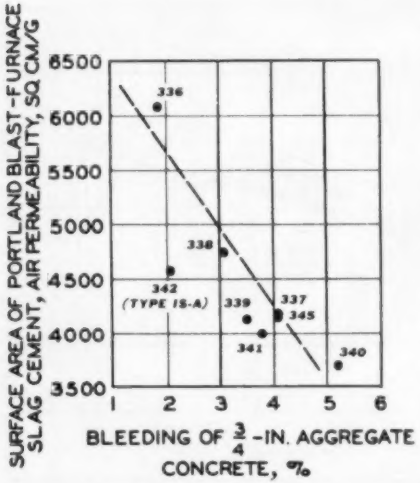

Fig. 8—Bleeding of concrete tends to vary inversely with surface area of cement

Bleeding

Fig. 8 indicates that there is a tendency for bleeding of the ¾-in. aggregate concrete mixtures to vary inversely with the surface areas of the cements.

Subsidence at early ages

Two 3-in. diameter cylinders approximately 6 in. high from each batch were tested for subsidence after placement by means of apparatus capable of measuring the distance from a reference point above the specimen to the top of the concrete. Specimens made from the first round of batches were tested by a procedure that was later revised and improved. Average results for Rounds 2 and 3 at 1, 3, and 7 days are given in Table 11.

Ring shrinkage

Pastes and mortars were tested at the National Bureau of Standards by the ring-shrinkage method[18] (Table 12). The performance of the portland blast-furnace slag cements in this test is described as good as compared to the general performance level of portland cements previously tested at the National Bureau of Standards.

Effects of early termination of moist curing

A study was made to determine whether the properties of portland blast-furnace slag cement concrete are affected more adversely by early termination of moist curing than are those of concretes made with Type II portland cement. Cylinders, 3 x 6 in., and beams, 3½ x 4½ x 16 in., were moist cured in the molds 3 days. Half of the specimens were stored, immersed in lime water, until time of test; the other half were stored at 50 percent relative humidity until time of test or 28 days age, and thereafter in laboratory air. Tests include determination of compressive and flexural strength at 7, 28, and 90 days and 1 year; static Young's modulus on compression specimens at 28 days and 1 year; and abrasion resistance of the beams tested at 90 days.

Tests at 7, 28, and 90 days indicate that compressive and flexural strength of portland blast-furnace slag cement concrete was adversely affected by termination of moist curing after 3 days approximately in the same fashion as such termination affected the strength development of the reference Type II portland cement concrete. Drying from 3 to 7 days age had a marked effect on flexural strength but only a small effect on compressive strength. For both the portland and the portland blast-furnace slag cement, the mag-

TABLE 12—SHRINKAGE TEST RESULTS

Test data	Serial No. of cement							
	336	337	338	339	340	341	342	345
Paste specimens								
Ring breaking								
Time, hr:min	17:07	23:56	12:36	±23	21:31	21:17	19:14	19:08
Shrinkage at breaking, percent × 10^3	59	67	44	69	61	47	57	45
Shrinkage, percent × 10^3								
15 min	6	6	7	9	7	6	8	6
30 min	8	8	10	12	8	9	11	8
1 hr	10	9	13	14	9	11	12	9
3 hr	12	11	16	18	12	14	16	12
6 hr	15	14	22	23	16	19	23	17
24 hr	103	64	80	71	71	52	67	59
7 days	222	130	124	129	143	129	149	166
21 days	267	177	163	163	188	189	168	199
28 days	272	182	178	176	205	203	180	224
1:275 mortar specimens								
Ring breaking								
Time, days	6–7	*	*	*	*	*	*	*
Shrinkage at breaking, percent × 10^3	—	*	*	*	*	*	*	*
Date made, 1956	3/26	3/26	3/26	3/26	3/26	4/3	4/2	4/3
Shrinkage, percent × 10^3								
15 min	6	8	7	7	7	6	7	7
30 min	8	10	9	8	8	7	9	8
1 hr	9	11	11	10	9	8	10	8
3 hr	10	11	12	11	10	9	11	9
6 hr	11	11	13	12	10	10	12	10
24 hr	55	32	36	29	37	22	30	23
7 days	112	63	69	61	61	54	63	58
21 days	113	65	79	65	66	59	69	65

*Not broken.

nitude of the effect increased with age for compressive strength and decreased with age for flexural strength.

Effects of reduced-temperature curing

The effects of reduced temperature on strength development of portland blast-furnace slag cement were studied since it was considered possible that blast-furnace slag, being a less active hydraulic material than portland cement, might be more seriously affected by reduced temperatures. Nine 3- x 6-in. cylinders were made and stored at 40 ± 5 F until tested in groups of three at 3, 7, and 28 days age. The 28-day cylinders were also tested for Young's modulus of elasticity. The results of these tests and those of comparable tests of companion specimens from the same batches cured at 73.4 ± 2 F are given in Table 13.

The effects of reduced-temperature storage in reducing compressive strength below that which would be obtained at normal temperature are greatest at the earliest age and become less with time. At 3 days the strength of Type II portland cement concrete cured at 40 F is about ¼ that of similar concrete cured at 73 F; at 7 days about ½, and at 28 days about 2/3. The portland blast-furnace slag cement concretes show the same effect. At 3 days age four blast-furnace slag cement concretes are less affected and four more affected by reduced temperature than the portland. At 7 days all are more affected, the one showing the maximum effect having a strength not much greater than ¼ that of comparable concrete cured at 73 F. At 28 days,

TABLE 13—EFFECT OF REDUCED-TEMPERATURE CURING ON STRENGTH DEVELOPMENT

Test data	Type II portland cement 330	Commercial portland blast-furnace slag cement							Experimental high-MgO portland blast-furnace slag cement 336
		Type I-S						Type IS-A 342	
		337	338	339	340	341	345		
Normal-temperature curing (73.4 ± 2 F)									
Compressive strength, psi, 3 days	2210	1300	1910	1400	1620	1450	1500	1510	1810
7 days	2960	2350	3050	2250	2320	2600	2290	2330	2970
28 days	5020	4100	4360	3710	3550	4120	3390	3580	3790
Young's modulus, 28 days, psi × 10^{-6}	5.01	4.87	4.80	4.52	4.98	4.85	4.89	4.53	4.82
Reduced-temperature curing (40 ± 5F)*									
Compressive strength, psi, 3 days	570(26)	290(22)	530(28)	290(21)	300(19)	560(39)	520(35)	410(27)	280(15)
7 days	1590(54)	870(37)	1380(45)	910(40)	1020(44)	1330(51)	1180(52)	1050(45)	910(31)
28 days	3520(70)	2330(57)	3470(80)	2430(65)	2350(66)	3180(77)	2410(71)	2630(73)	2960(78)
Young's modulus, 28 days, psi × 10^{-6}	4.07(81)	3.40(70)	4.12(86)	3.64(81)	3.55(71)	4.14(85)	3.81(78)	3.79(84)	4.05(84)

*Values in parentheses indicate percentage of comparable results at 73.4 ± 2 F curing.

five show less effect and three show more effect. It is suggested that the effects of reduced storage temperatures probably affect the rate of strength gain of portland blast-furnace slag cement concretes to differing degrees, but only to the same extent and in essentially the same fashion as such conditions variously affect concretes made with different portland cements.

Resistance to accelerated freezing and thawing

Three 3½- x 4½- x 16-in. beams were made from each batch, moist cured to an age of 14 days, and tested* for resistance to accelerated laboratory freezing and thawing in water (Table 11). These results (DFE_{300}), ranging from 34 to 53 for the concretes made with the portland blast-furnace slag cements and 51 for the concrete made with Type II portland cement, are typical of results for an aggregate combination such as was used in these tests (crushed limestone coarse aggregate with natural siliceous sand containing some porous chert in the coarser sizes), and are somewhat lower than would have been expected had manufactured limestone sand been used.

Resistance to field exposure

Six 3½- x 4½- x 16-in. specimens were made from each batch and moist cured to an age of 14 days. Three of these have been installed at Treat Island, Me., and the other three at St. Augustine, Fla.

Bond to and corrosion of reinforcing steel

Bond pull-out specimens were tested at 28 and 90 days, and specimens containing completely embedded steel bars were made for observation of corrosion of steel. These specimens were made with the two portland blast-furnace slag cements of greatest sulfide sulfur content, 338 and 339, and the reference Type II portland cement only. The bond resistance to pull-out of concretes made with portland blast-furnace slag cements was what would be expected of portland cement concretes of comparable compressive strength. The corrosion specimens each contained four pieces of steel, two with 1 in. of cover and two with 3 in. of cover. One group of these specimens was broken and the steel examined for corrosion after 90 days moist curing in the laboratory and others will be so tested after 1 year of such curing and 2, 4, and 5 years exposure to sea water at St. Augustine. A count of rust spots, weighted as to their size, indicates that at 90 days the steel in concrete made with Cement 338 had 28 percent and that made with Cement 339 had 75 percent as much rusted area as that in concrete made with reference Type II portland cement. Pearson[19] has commented on the rusting of reinforcing steel in concrete made with blast-furnace slag as follows: "Discussion often occurs as to the harmful effects of the sulfur (in slag) . . . the opinion is frequently expressed that the sulfur, which is mostly combined as sulfide, can be oxidized to free sulfuric acid and damage to steel or other metal may result. It is further assumed that the calcium sulfide would be oxidized to calcium sulfate and then lead to gypsum swelling in the concrete.

*Test method used, CRD-C 114 of the Corps of Engineers' *Handbook for Concrete and Cement*, is essentially similar to that described in ASTM C 290.

These opinions are based on bad experience with the older type, unsatisfactory slags. Detailed investigations, the results of which have been confirmed further by practical experiences, at the Berlin-Dahlem Material Testing Station (*Stahl und Eisen*, V. 37, pp. 616-632) clearly served to show that blast-furnace slag has no influence on the rusting of steel in concrete. If rust appearances have been noticed with reinforced concrete construction when slag has been used . . . then this can be ascribed to unsatisfactory bedment of the reinforcing."

Abrasion resistance

The halves of the 3½- x 4½- x 16-in. beams that had been tested for flexural strength at 90 days were broken in half; the two end pieces of each beam were air dried for 7 days, and two 2-sq-in. areas on each molded side face of each piece were tested for abrasion resistance. The test consisted of blowing 300 ± 25 g of No. 30 to No. 50 size crushed steel shot at the test area from a distance of 4 in. by an air pressure of 60 psi for a period of 30 sec. After the four test areas had been abraded the loss in weight was determined. Since there were three beams from each batch tested at 90 days, there were 72 test spots or a total area of 1 sq ft based on eighteen determinations of weight loss for each concrete mixture (Table 11). The abrasion loss values ranged from 101 to 133 g per sq ft and do not appear to be significantly correlated with other properties of the concrete.

Length change

Six 2- x 2- x 11-in. prisms were made from each batch; three were tested for shrinkage on drying according to CRD-C 56;[6] the other three were tested for expansion caused by continued immersion in water. Length change (shrinkage and expansion) averages at 180 days are given in Table 11.

Color

The hydration products in concrete made with hydraulic blast-furnace slags and portland blast-furnace slag cements develop a bluish-green color internally when stored damp. Candlot[20] investigated this phenomenon in 1898 and concluded that it is due to calcium sulfide in the cement and stated that it was especially noticeable in a sample kept in sea water and that it fades on exposure to dry air. The Board of Engineer Officers in its 1902 report to the Chief of Engineers[3] noted that slag cement is characterized by "the intense bluish-green color in the fresh fracture after long submersion in water, due to the presence of sulphides, which color fades after exposure to air." They also add: "The oxidation of sulphides in dry air is destructive" of slag-cement mortars and concretes so exposed. Current specifications limit the amount of sulfide sulfur that may be present in portland blast-furnace slag cements to 2.0 percent.

Mortar cubes after test for compressive strength, and concrete prisms after test for resistance to accelerated freezing and thawing were examined

TABLE 14—SULFIDE SULFUR AND IRON CONTENT AS RELATED TO COLOR DEVELOPMENT

Serial No.	Sulfide sulfur, percent		Fe_2O_3, percent in slag	Intensity of color development on concrete*
	In slag	In portland blast-furnace slag cement		
345	0.77	0.28	0.41	B
341	0.98	0.28	0.43	C
340	1.29	0.36	0.42	D (least)
337	1.10	0.42	0.38	B
342	1.11	0.47	1.23	C
336	1.49	0.63	1.05	A (greatest)
339	1.51	0.67	2.07	B
338	1.68	0.73	1.37	B

*Greatest intensity of color development was for 336, designated A; least intensity of development was for 340, designated D. Color development for 337, 338, 339, and 345 was similar, designated B, and of somewhat greater intensity than for 341 and 342, which were similar to each other, designated C.

for development of bluish coloration by stereoscopic microscope and by petrographic microscope in immersion mounts and thin sections; also polished sections were examined in reflected light. No unusual reaction or hydration product was observed, and the only positive finding was the existence of color differences from inside to outside of cubes, and from cement to cement. The color development observed on the freshly fractured concrete surfaces was similar to that observed on fracture surfaces of the mortar cubes. Color development was strongest on surfaces made with Cement 336 and least on those made with Cement 340. Color intensity on surfaces made with Cements 337, 338, 339, and 345 were similar and somewhat more intense than that on surfaces of specimens made with Cements 341 and 342. The color classed according to the National Research Council Rock Color Chart ranged from dark bluish green to light bluish green (chart symbols 5 BG 3/2 to 5BG 5/2 or 5 BG 5/1). The area of a transverse fracture in a concrete specimen that was bluish green when examined immediately after breaking was greatest for Cement 336, the colored area in specimens of which ranged from the entire surface to a central area about 2 x 3 in. Scattered colored spots were common in the central areas on fractured surfaces of specimens made with all other cements. No relation was noted between area of intensity of color and physical condition of the concrete specimens. Only those specimens made with Cement 336 showed any development of blue-green color on exterior surfaces. The color only developed in moist-stored specimens and faded and disappeared when the specimens dry out. Early workers[20,2] assumed that the constituent of the cement responsible for the color phenomenon was the sulfide present in the slag. The slags used in making the cements and the cements used in this investigation were found on chemical analysis to contain sulfide sulfur as shown in Table 14.

The portland blast-furnace slag cements containing less than 0.45 percent sulfide sulfur were made with slags containing less than 0.5 percent Fe_2O_3;

those cements with more than 0.45 percent sulfide sulfur were made with slags containing more than 1.0 percent Fe_2O_3. On the hypothesis that the color was caused by iron salts in a reduced state—reduction produced perhaps by the action of sulfides—selected, freshly fractured surfaces showing well-developed blue-green coloration were treated with an oxidizing agent (hydrogen peroxide) and others with a reducing agent (stannous chloride, 3-molar solution in hydrochloric acid). After 5 min the reducing agent had produced no visible change; the oxidizing agent had produced a marked bleaching (fading) of the color.

Since concrete containing Slag 336 showed greatest color development, small amounts of ground Slag 336 and the portland blast-furnace slag Cement 336 were stored in water for 2 months. The slag developed no set and no blue-green color. The cement set and developed blue-green color after 3 days, the color being progressively darker from top to bottom of the material under water. Slag 336 was also stored in sodium hydroxide and ammonium hydroxide, and behaved as in water: no set and no blue-green color. Samples of Slag 341 were stored in water for 2 months; they hardened within the first week and developed signs of blue-green color after 17 days. After 2 months the color was more green than blue-green. From these observations it is suggested that: (a) The degree to which blue-green color is developed on freshly fractured surfaces of moist-stored mortar and concrete made with portland blast-furnace slag cements varies from cement to cement; it may be related in part to sulfide sulfur content, but it is not found directly related to sulfide sulfur content as indicated from chemical analysis. (b) The coloring agent is apparently in the reduced state and its color is lost by oxidation. (c) Some slags will set and develop blue-green color when stored alone in water; others will not. One slag that did not set and change color was used in a portland blast-furnace slag cement which, when so stored, did set and change color. However, this study did not reveal the nature of the constituents and reactions responsible for the color.

Rating

A rating system for laboratory performance tests on structural concrete has been developed,[12] based on bleeding of freshly mixed concrete, permeability, resistance to freezing and thawing, drying shrinkage, and compressive strength at 28 and 90 days. Applying this system to the results of the tests (Table 11) with the exception that the rating value for resistance to freezing and thawing was reduced from 60 to 40 for the reasons mentioned in discussion of the results of that test, seven of the mixtures gave ratings of 100; these included those made with reference Type II portland cement and with portland blast-furnace slag Cements 337, 338, 339, 341, 342, and 336. One, with Cement 340, rated 90, since its bleeding, 5.2 percent, exceeded the rating value of 5.0; one, with Cement 345, rated 65, since its value for resistance to freezing and thawing, 34, failed to meet the rating value of 40. All mixtures had permeabilities and drying-shrinkage results well below the rating values of 25 (K_c) and 0.070 percent respectively, and compressive

TABLE 15—RESULTS OF TESTS OF 6-IN.-AGGREGATE CONCRETE MADE WITH PORTLAND CEMENT AND WITH PORTLAND BLAST-FURNACE SLAG CEMENTS

Test data	Type II portland cement 330	Commercial portland blast-furnace slag cement							Experimental high-MgO portland blast-furnace slag cement 336
		Type I-S						Type IS-A 342	
		337	338	339	340	341	345		
Temperature rise, deg F,									
3 days	12.8	12.6	15.2	13.7	13.9	13.0	11.5	17.1	12.1
7 days	19.4	15.0	20.7	18.8	19.5	19.5	13.7	21.8	17.8
28 days	20.5	18.3	22.7	22.6	23.1	22.1	14.4	23.0	19.8
Diffusivity, sq ft/hr	0.054	0.050	0.050	0.049	0.049	0.049	0.047	0.048	0.048
Specific heat, B/lb/F	0.20	0.21	0.20	0.20	0.20	0.20	0.20	0.20	0.18
Compressive strength, psi,									
7 days	1540	1160	1570	1260	1250	1110	1310	1130	1400
28 days	2710	2220	2670	2600	2110	1950	2280	2070	2230
Young's modulus, psi $\times 10^{-6}$, 28 days	3.96	3.85	4.13	4.08	3.51	3.42	3.82	3.52	3.99
Air content, percent	3.2	3.2	3.0	3.2	3.1	3.2	3.2	3.3	3.1
NVR, ml/bag	65	47	50	43	40	37	47	18	43
Bleeding, percent	6.1	12.9	7.8	4.8	8.2	1.6	5.6	4.2	3.4

strengths at 28 and 90 days above the rating values of 3000 and 4000 psi respectively.

LARGE-AGGREGATE CONCRETE MIXTURES

Nine mixtures using the same cements and blends that were used with the small aggregate were made with crushed limestone aggregates graded up to 6 in. in size. These mixtures had an air content of 3.0 ± 0.5 percent, a cement content of 2.25 bags per cu yd, and a slump of $1\frac{1}{2} \pm \frac{1}{2}$ in. Specimens made from these mixtures had a temperature of 49 ± 2 F immediately after placement. One 2-ft cube and six 3- x 6-in. cylinders were placed from each mixture. The concrete used in casting the cylinders was wet-sieved through a $\frac{3}{4}$-in. sieve. The cubes were tested for adiabatic temperature rise and thermal diffusivity. A sample of concrete from each cube was tested for specific heat. Three cylinders from each mixture were tested for compressive strength at 7 and 28 days. Data obtained in all of these tests are given in Table 15. Calculated relationships between determined adiabatic temperature rise and determined heat of hydration of the cements are given in Table 7.

SPECIFICATIONS AND RECOMMENDATIONS

An examination of the federal and ASTM specifications for portland blast-furnace slag cement indicates that the product so specified is intended to have essentially the same minimum performance requirements as Type I portland cement. A comparison of the specification requirements for Types I and II portland cement and Type IS portland blast-furnace slag cement as given in federal specifications SS-C-192b and SS-C-197 (Table 16) reveals that a number of requirements are identical, and that a few requirements apply to Type IS but not Type II (manganese, sulfide sulfur, and

TABLE 16—FEDERAL SPECIFICATION REQUIREMENTS FOR PORTLAND AND FOR PORTLAND BLAST-FURNACE SLAG CEMENTS

Requirement	Fed specification SS-C-192b		Fed specification SS-C-197
	Type I	Type II	Type IS
Chemical Limitations			
Slag content, percent	—	—	25 to 65
MgO, percent, maximum	5.0	5.0	5.0
SO_3, percent, maximum (C_3A 8 percent or less)	2.5	2.5	
($C_3A > 8$ percent)	3.0	—	2.5
Mn_2O_3, percent, maximum	—	—	1.5
Sulfide sulfur, percent, maximum	—	—	2.0
Insoluble residue, percent, maximum	0.75	0.75	1.0
Loss on ignition, percent, maximum	3.0	3.0	3.0
Minimum slag ratio: $\frac{CaO + MgO + 1/3\ Al_2O_3}{SiO_2 + 2/3\ Al_2O_3}$	—	—	1
Clinker: Al_2O_3, maximum, percent	7.5*	6.0*	7.5
Fe_2O_3, maximum, percent	6.0*	6.0*	6.0
MgO, maximum, percent	—	—	5.0
C_3A, maximum, percent	15 *	8 *	15
SiO_2, percent, minimum	—	21.0	—
$C_3S + C_2A$, percent, maximum	—	58	—
Physical Tests			
Retained on No. 325 sieve, percent, maximum	—	—	12
A.P. fineness, sq cm/g, average minimum	2800	2800	3400
one, minimum	2600	2600	3200
Autoclave, expansion, percent, maximum	0.50	0.50	0.20
Gillmore, initial, minutes, minimum	60	60	60
final, hr, maximum	10	10	10
Air content, mortar, percent, maximum	12.0	12.0	12
Compressive strength, psi, 3 days, minimum	1200	850	900
7 days, minimum	2100	1600	1800
28 days, minimum	3500	3000	3000
Heat of hydration, 7 days, cal/g, maximum	—	70	—
28 days	—	80	—

*Limit is on the cement, rather than on the clinker.

coarser than No. 325 sieve). Several items are more restrictive in the case of IS than II, and there is only one for which the reverse is the case (insoluble residue: II: 0.75 percent maximum; IS: 1.0 percent maximum). Limits imposed on Type II that are not imposed on IS relate to tricalcium aluminate content and heat of hydration.

Type II portland cement is intended to differ from Type I in being required to comply with certain limitations designed to insure a moderate heat of hydration and a moderate sulfate resistance. Type I cement may, and frequently does, have these characteristics but their presence is not mandatory.

Portland blast-furnace slag cement meeting requirements for Type IS may be regarded as the essential equivalent of Type I, or Type II portland except for sulfate resistance and heat of hydration, since the specifications require performance essentially equivalent to Type I and Type I differs from Type II only with respect to sulfate resistance and heat of hydration.

The results of these tests indicate that portland blast-furnace slag cements

that comply with current federal and ASTM requirements may be considered essentially equivalent to Type I portland cement. Like Type I portland cement, such portland blast-furnace slag cement may possess the additional properties that distinguish Type II portland cements, i.e., moderate heat of hydration and moderate sulfate resistance. However, like Type I and Type I-A portland cements, Type IS and IS-A portland blast-furnace slag cements are not required to possess these qualities. To insure that portland blast-furnace slag cements possess, in addition to the qualities now required of them, the distinguishing qualities of Type II portland cements, it may be necessary that the portland-cement clinker component have the distinguishing qualities of Type II clinker.

From the practical standpoint of specification writing and determining compliance with specification requirements, it would be preferable to require that the finished portland blast-furnace slag cements meet performance requirements for heat of hydration and sulfate resistance rather than to require that its clinker constituent meet chemical requirements. It has therefore been suggested that the heat of hydration requirement be that now required for Type II portland in SS-C-192a and that efforts be made to develop a suitable acceptance performance test for sulfate resistance. In the meantime assurance of moderate sulfate resistance may be provided by requiring that the portland-cement clinker component of the portland blast-furnace slag cement not contain more than 8 percent C_3A. When only the moderate heat of hydration characteristic is required, only that limit need be added to the specification. When only moderate sulfate resistance is required in addition to properties now controlled, only the C_3A limit on the clinker need be added. When both additional characteristics are required, both requirements may be added.

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Title No. 54-14

Destructive Impulse Loading of Reinforced Concrete Beams*

By F. T. MAVIS† and M. J. GREAVES‡

SYNOPSIS

Reinforced concrete beams, identical except for grade of reinforcement, were tested by pairs under destructive impulse loads in a unique spring-powered testing machine. Data from each dynamic test were filmed by a high-speed motion picture camera which superimposed continuous oscilloscope records of load and reaction, and photographs of the beams at frequency of 1000 pictures a second. Data such as deflection of beams, crack formation, and time intervals, were also recorded on the motion picture film for repeated projection, microscopic frame-by-frame study, and analysis.

Typical tests are detailed in this paper and dynamic behavior phases are identified and interpreted. Differences in behavior of beams under dynamic and static loadings are discussed further under headings of crack pattern and "whiplash", shear distribution and bond failure, steel strain and energy absorption. Bibliography lists 116 pertinent references.

INTRODUCTION

Structural or intermediate-grade steel is more ductile than hard-grade steel as determined by standard coupon tests. This fact may have supported the opinion that structural-grade or intermediate-grade steel is also better than hard-grade steel as reinforcement to resist heavy blast forces and destructive impulse loads. This opinion is reported to have the support of tests.§ Yet in an exhaustive search of published literature the authors did not find experimental data on tests of reinforced concrete beams that would seem to warrant such an opinion.

Accordingly, the authors carried out a program of studies as summarized in this paper, and arrived at conclusions which differ from the opinions referred to above.

Experimental data for each dynamic test were recorded by a high-speed motion picture camera which photographed simultaneously the behavior of

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§For example, quoting from U. S. Atomic Energy Commission, *The Effects of Atomic Weapons*, U. S. Government Printing Office, Washington, D. C., 1950, p. 380: "Tests have shown that in this respect [resistance of buildings to blast damage] a structural or intermediate-grade steel is much better for reinforcement than a hard-grade steel. When a reinforced concrete structure is subjected to heavy blast forces and damage occurs, the softer steel may elongate or deform without failure whereas the hard-grade steel snaps. More energy is absorbed in the plastic range by the structural or intermediate-grade steel. In the usual case such a selection will have little effect on rigidity."

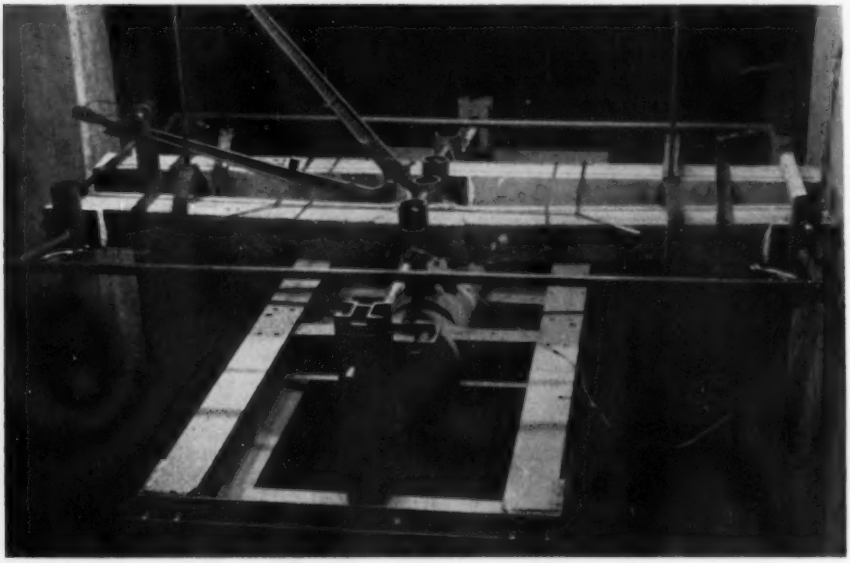

Fig. 1—Testing machine assembled

two beams dynamically loaded by the same impulse, and oscilloscope traces of load and reaction as sensed by electric resistance strain gages. Data so recorded are well suited to microscopic study and restudy frame by frame.

The authors observed that in every paired test where a beam with hard-grade steel reinforcement was dynamically loaded opposite an otherwise identical beam with intermediate or structural-grade* steel reinforcement, the beam reinforced with hard-grade steel outperformed the otherwise identical beam reinforced with intermediate-grade or structural-grade steel. They concluded from their study that the rate of loading is a highly significant variable influencing reactions, stress distribution, limiting strain, and load capacity of a reinforced concrete beam. In none of the authors' tests of reinforced concrete beams was structural* or intermediate-grade steel better for reinforcement than a hard-grade steel. In no test did the hard-grade steel snap. In different tests where damage after dynamic loading was comparable, nearly the same energy was absorbed in the plastic range by hard-grade steel as by intermediate-grade steel. However, about 13 percent more initial load was required to start the damage in a hard-grade beam than in an intermediate-grade beam.

TEST PROCEDURES

Beam specimens

Beams tested in this program were identical except for grade of reinforcement. They were 4.1 in. wide, 5.9 in. deep, and 78 in. long. Depth to the center of the steel was 4.6 in. Each beam was reinforced with one #5 bar either of intermediate-grade steel or hard-grade steel.

*For comparative tests of beams with structural-grade steel and hard-grade steel see F. T. Mavis and F. A. Richards, "Impulse Testing of Concrete Beams," ACI JOURNAL, Sept. 1955, *Proc.* V. 52, pp. 93-102.

The beams were cast in wooden gang molds with intermediate-grade bars and hard-grade bars alternating. The bars were placed near the top surface and held securely in position at ends by slots in the forms, and at the third-points by twisted wires while the concrete was cast. The position of each bar was checked before filling the forms, and verified when the beams were broken up after the test.

The deformations on both grades of steel bars were identical and conformed to ASTM specification A 305. The intermediate-grade steel had a yield point of 40,000 psi and an ultimate of 70,000 psi as determined by standard laboratory tests. Elongation in 8-in. gage was 25 percent. For the hard-grade bars the yield point was 75,000 psi and the ultimate 130,000 psi, with 10 percent elongation in 8 in.

Concrete for all beams came from one batch in a transit-mix truck. The concrete at 90 days had an average ultimate strength of 6500 psi and an average secant (to $0.45f_c'$) modulus of elasticity of 4.1×10^6 psi.

Apparatus

Beams were tested dynamically in pairs in a specially designed machine. Instrumentation and method are summarized briefly; the details are beyond the scope of this paper.

The testing machine (Fig. 1) is diagrammed in Fig. 2. A helical spring (1) was compressed between two needle beams (2) tied together at the bottom by a yoke (3), and at the top by a "chain" made up of two load bars (4) and a notched link (5). This was called the "machine circuit." In parallel with the notched link, the two test beams, (6) and (7), were tied toegther at their ends by reaction yokes (8). This was the "test circuit."

In preparing to test, the spring (1) was loaded and the machine circuit, consisting of needle beams (2), yoke (3), load bars (4), and notched link (5), was stressed. There was no load on the beams (6) and (7). Just before a test the reaction yokes (8) were tightened to about 500 lb each to take up slack in the system. During a test when the notched link (5) was cut by the bolt cutter (shown open in Fig. 1), the initial force in the link of the machine circuit was *suddenly* transferred to the beams of the test circuit.

The test beams were cradled in stirrups (shown in Fig. 1) suspended by long rods. The needle beams (2) were held at the bottom by a fixed yoke (3). There was complete freedom of movement in the plane of the test beams since only the tops of the suspender rods and the bottoms of the needle beams (2) were restrained against the linear displacement, but they were free to rotate.

Data from dynamic loading tests were recorded by an Eastman high-speed motion picture camera and oscilloscopes. Loads at midspan of the test beams were indicated by SR-4 gages attached to load bars (4) (Fig. 2). Reactions at the end of the beams were indicated by SR-4 gages attached to bars of the reaction yoke (8).

In the field of the *side* lens of the camera were two oscilloscopes, so placed that one directly and the other by mirror, recorded load and reaction data in a continuous trace through the *back* of the motion picture film. Simultaneously, pictures of the test, including deflection and crack data, were photographed through the *front* lens at about 1000 pictures a second.

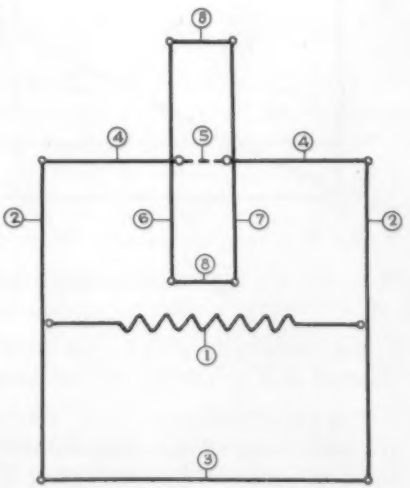

Fig. 2—Schematic diagram of impulse testing machine

TEST RESULTS

The testing program involved collecting in a few milliseconds a complete record of load, reaction, and displacement for each beam as a function of time. Beams were tested in pairs—an intermediate-grade beam opposite a hard-grade beam, or a hard-grade beam opposite a steel beam of equal weight made of steel channels boxed and welded. In each test a "weaker" beam was tested opposite a "stronger" beam of equal weight.

Early in the program, simultaneous load and reaction data were indicated as a trace on a biaxial oscilloscope. This trace was photographed with another motion picture camera at 64 frames per second. A typical trace is shown in Fig. 3, with load on the beam as abscissa and reaction as ordinate. This trace is for the test of an intermediate-grade beam (I-6) opposite a hard-grade beam (R-24) when the initial link load was 6.5 kips and the initial beam reaction totaled 1.0 kip. To see what happened during the test, begin at the lower right end of the curve—at a load of 6.5 kips and a reaction of 1.0 kip. Follow the contortions of load and reaction defined by the line which finally indicates static equilibrium at the upper end of the dotted line where both load and reaction are 3.8 kips. Thereafter, load and reaction are statically

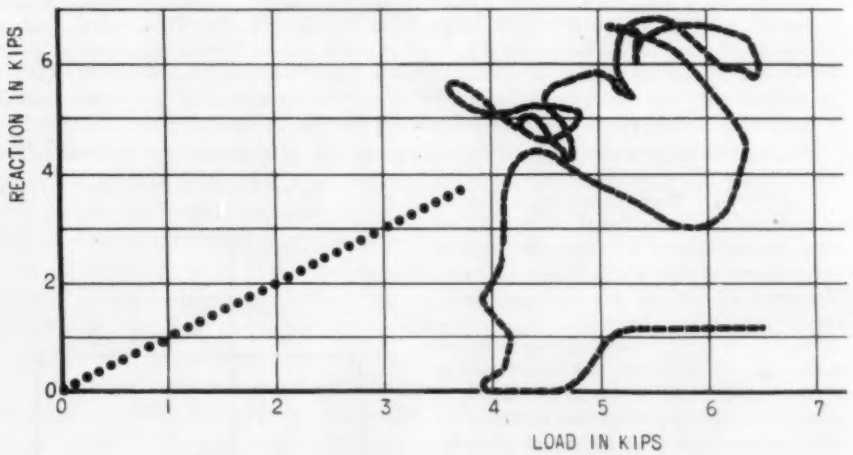

Fig. 3—Load-reaction curves for 6.5-kip test (I-6, R-24)

in balance as indicated by the dotted line which reached zero linearly as the residual load of 3.8 kips was released after the dynamic test ended.

Two facts illustrated in Fig. 3 were typical in these tests: (1) both load and reaction during the dynamic test reached maximums which were nearly double those at which intermediate-grade beams failed in a static test; and (2) load and reaction during a dynamic test are unequal except when the dynamic test curve crosses the extension of the straight dotted diagonal line.

Fig. 4 shows consecutive pictures from the high-speed motion picture record of a typical test. Each frame shows what happened in about one millisecond. The dotted line along the right shows how load varied, and the dotted line at the left shows variation in reaction. One half the width of the motion picture frame represents about 10,000 lb. The high-speed motion picture record was enlarged more than 100 diameters by projection, and pertinent data were traced on sheets of drawing paper. In this way the oscillographic record was projected and analyzed from the 16-mm film to a time scale of 1000 yd per sec. In effect, then, an oscillograph traced its record at a speed faster than a rifle bullet as projected. Simultaneously the camera made 1000 complete photographic records every second during a test.

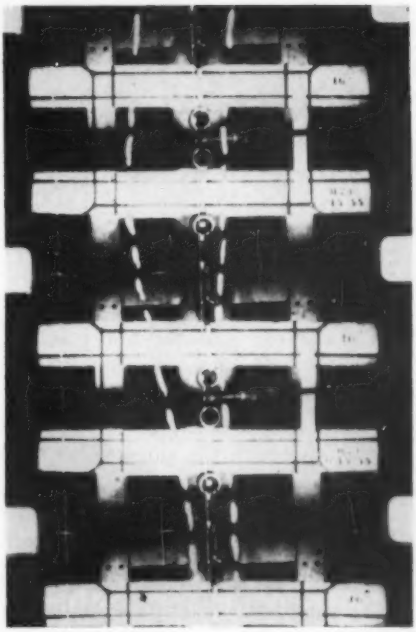

Fig. 4—Pictures from a typical high-speed camera record

The dotted curve in Fig. 5 shows how load on the beam varied with time during a test of an intermediate-grade beam (I-6) and a hard-grade beam (R-24). At the instant the trigger link was cut, the initial load, 6.5 kips, was transferred from the testimg-machine spring to the beams. After pulsing to maximums of about 6.5 kips at 7.5 and 15 milliseconds, the load decayed fairly regularly to its residual asymptote of 3.8 kips.

The dotted curve in Fig. 6 shows how reaction varied with time during the same test. After the load was suddenly applied, the initial reaction of 1.0 kip dropped to zero. Then the reaction surged to maximums of about 6.5 kips (the same as the initial load) at 10, 13, and 18 milliseconds. Thereafter, the reaction also decayed fairly regularly to its residual asymptote of 3.8 kips.

In Fig. 5, the solid curve marked P is represented by the formula at the upper right. According to the formula, the initial load (at time $T = 0$) is $(3750 + 2750) = 6500$ lb. As T increases, the load P asymptotically approaches 3750 lb, which is the residual load on the beam after the test.

In Fig. 6, the solid curve is represented by the formula for reaction R shown in the upper right of the figure. According to the formula the reaction at the instant the impulse is applied, namely $T = 0$, is zero. It reaches a maximum when $T = 0.0135$ sec, or where $T^* = 1$. Thereafter it decays to the residual reaction, 3750 lb, after T becomes large.

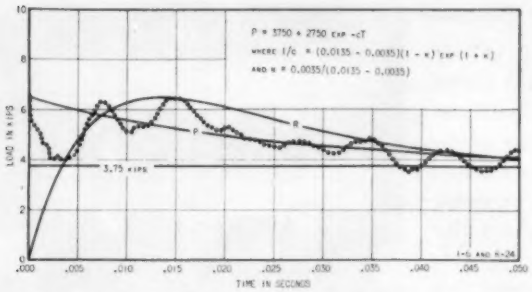

Fig. 5—Load-time curve for a 6.5-kip test (I-6, R-24)

The curves for load P and reaction R are shown in Fig. 5 by the solid lines, and their common asymptote, 3.75 kips, is also shown. The formulas for load and reaction are related so that: (1) their maximum ordinates are the same; (2) they are asymptotic to the same ordinate; (3) the areas under the two curves approach the same value (total impulse) as the time T increases; and (4) they have a common invariant time value, $T = 0.0135$ sec, which inheres in the particular test set-up but does not depend on the load applied nor on the extent of damage to the beams.

The interval between the time coordinates of the centroids of the load-impulse and the reaction impulse may be of interest. These coordinates are easily computed:

time coordinate of centroid of the load impulse $= 0.0135\ e = 0.0135 \times 2.718 \ldots = 0.037$ sec
time coordinate of centroid of the reaction impulse $= 0.0135 \times 2 = 0.027$ sec

Accordingly in these tests, the total load impulse leads the total reaction impulse by $0.0135\ (e - 2) = 0.01$ sec.

Data from the authors' tests, of which the dotted curves of Fig. 5 and 6 are representative, agree substantially with the criteria outlined in the last two paragraphs.

Fig. 7 shows deflection as a function of time for the test of beams I-6 and R-24. During the first 10 milliseconds, both beams deflected equally. At $T = 0.008$ sec both load and reaction were 6.0 kips, as can be read from Fig. 5 and 6. Beam R-24 did not fail during the test. Since R-24 and I-6 deflected

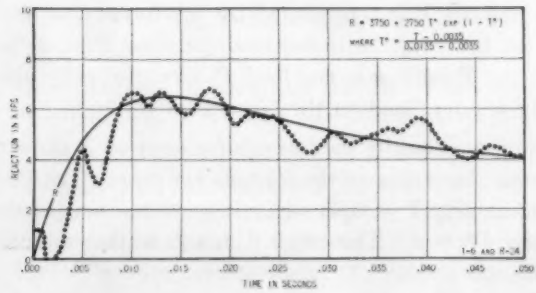

Fig. 6—Reaction-time curve for a 6.5-kip test (I-6, R-24)

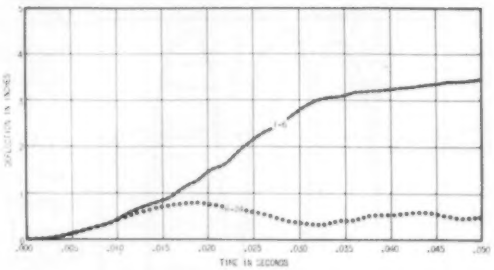

Fig. 7—Deflection-time curves for an 6.5-kip test (I-6, R-24)

the same during the first 10 milliseconds of the test and there was no unbalanced accelerating force at $T = 0.008$ sec, the inference seems reasonable that both beams withstood instantaneous loads of 6.0 kips dynamically with equal damage. Yet in all static tests the ultimate load on intermediate-grade beams was about 3.6 kips and on hard-grade beams about 7.0 kips. Therefore momentarily the intermediate-grade beam resisted by bending, while it was *dynamically* in equilibrium, a load about ⅔ greater than the static ultimate load capacity of the beam. In other tests the dynamic ultimate load was nearly twice the static ultimate load.

Momentarily during the "undamaged" interval of 8 milliseconds the calculated stresses in the steel of both beams approached twice the static yield point of the intermediate-grade steel and 1⅛ times its static ultimate strength. Moreover this stress level was consistent with measurements of elastic curvature made independently from the high-speed motion picture films.

Static-dynamic comparisons

Fig. 8 shows the performance of two intermediate-grade beams, one (I-2) loaded statically in a universal testing machine and the other (I-5) dynamically in the spring-loaded machine. The static load-deflection curve was linear to 3 kips, reached a maximum of 3.6 kips, then dropped off to 2.0 kips before the beam failed at a deflection of 4 in.

The dotted curve of Fig. 8 shows dynamic load-deflection measurements for a similar intermediate-grade beam. The maximum load applied dynamically was 7.5 kips—about twice the maximum of 3.6 kips in the static test. It follows that the dynamic stresses must also have been about twice as large as

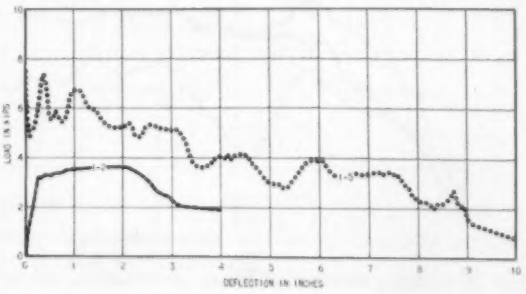

Fig. 8—Load-deflection curves for destructive tests of two intermediate - grade beams: I-2 static; I-5 dynamic

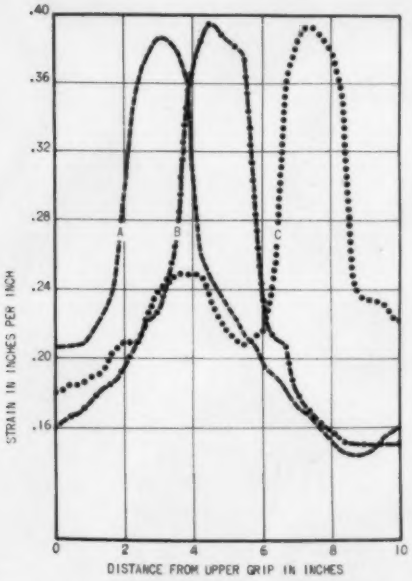

Fig. 9—Strain distribution along intermediate-grade bars after static tension tests

the static stresses. The beam was supporting loads dynamically through a deflection range about 2.5 times the deflection range of the static test.

Fig. 9 shows strain distribution in a 2-in. gage length along intermediate-grade bars after static tension tests, and Fig. 10 shows similar data for bars taken from beams after dynamic tests. It is clear by comparing Fig. 9 and 10 that only a small fraction of the available ductility of the intermediate-grade reinforcing steel as indicated by the static tension tests was effective in the dynamically loaded beam.

Fig. 11 shows strain distribution along hard-grade reinforcing bars after static tension tests. Since beams reinforced with hard-grade bars outperformed beams reinforced with intermediate-grade bars both statically and dynamically, it should be evident by comparing Fig. 9 and 11 that the greater ductility of the intermediate-grade steel was no advantage to intermediate-grade steel as beam reinforcement. This is discussed further in another section of this paper.

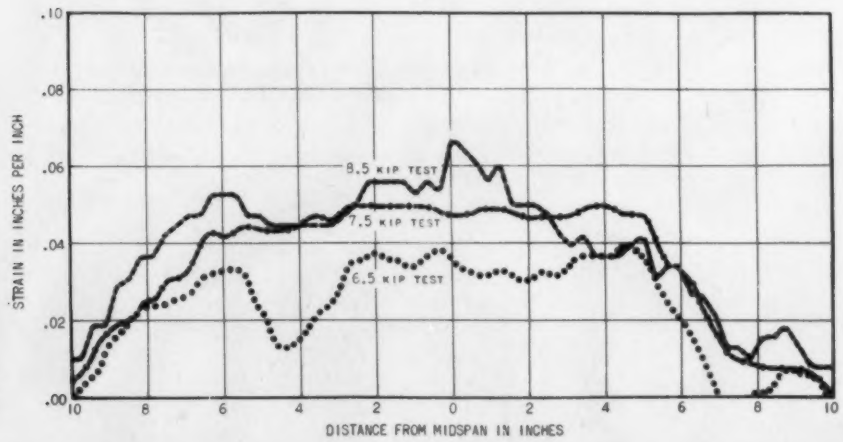

Fig. 10—Strain distribution along intermediate-grade bars after dynamic beam tests

Fig. 12 shows a similar comparison for two hard-grade beams, R-121 tested statically and R-126 tested dynamically. The dashed curve, which applies to a static test, shows that the hard-grade beam was elastic over a load range of 6 kips, or twice that of the intermediate-grade beam I-2.

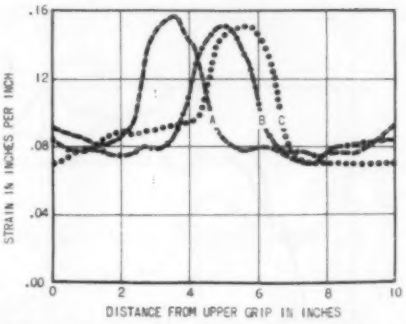

Fig. 11—Strain distribution along hard-grade bars after static tension tests

The stress in the concrete was near ultimate when the elastic limit of the hard-grade steel was reached. When the concrete disintegrated, beam R-121 failed after little inelastic yielding of steel at a deflection of 1.2 in. However, the steel was intact and had at this time undergone only a small fraction of the elongation possible even for hard-grade steel.

The dotted curve of Fig. 12 for a dynamic test of hard-grade beam R-126 shows that the beam supported loads until the deflection was well over 10 in., or about ten times the maximum deflection in the static test.

Much more energy is required to break a beam in a dynamic test than in a static test; and the relative amount is even greater for hard-grade beams than for intermediate-grade beams. Differences in behavior of two identical beams statically and dynamically have been the focus of many questions and comments. The comment was often heard that the weaker beam in a pairwise test "protects" the stronger beam; and that perhaps if a hard-grade beam were tested against a still stronger beam the hard-grade beam, being the weaker of the two, would fail. Accordingly, the authors tested reinforced concrete

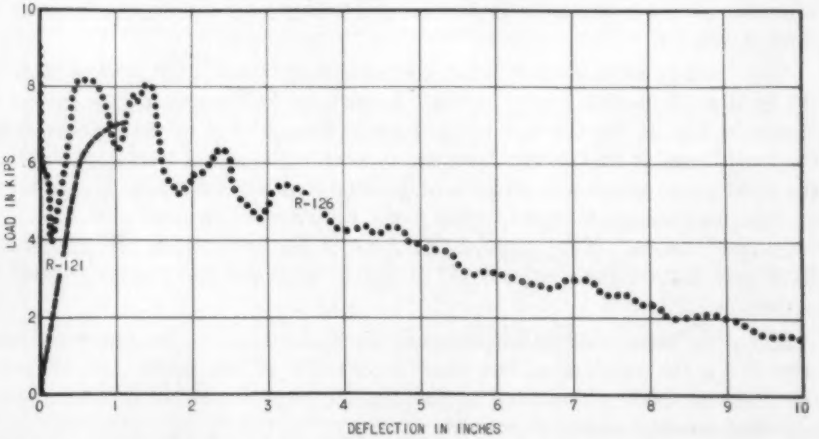

Fig. 12—Load-deflection curves for destructive tests of two hard-grade beams: R-121, static; R-126, dynamic

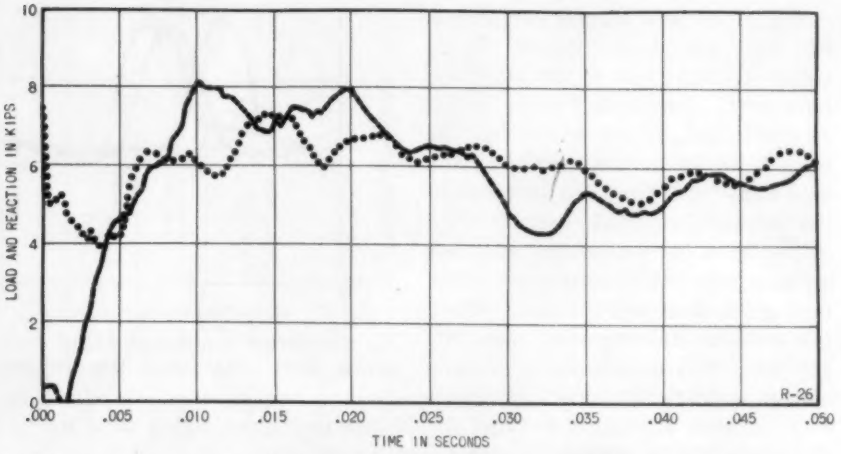

Fig. 13—Load-time and reaction-time curves for 7.5 kips suddenly applied to a hard-grade beam

beams opposite a beam of boxed steel channels which had the same mass as a concrete beam. This steel beam weighed the same as the concrete beam but it was stiffer and stronger.

Tested pairwise with the steel beam at an energy level of 40 in.-kips, corresponding to an initial load of 7500 lb, the maximum deflection of a hard-grade beam was 1 in. and its permanent deflection was only 0.5 in. At an energy level of 40-in. kips, corresponding to an initial load of 7500 lb, intermediate-grade beams were regularly destroyed. Load and reaction curves for the test of R-26, a hard-grade beam, opposite the stiffer and stronger steel beam are shown in Fig. 13. Deflection and velocity are shown as a function of time in Fig. 14.

At an energy level of 60 in.-kips, corresponding to an initial load of 9000 lb, the hard-grade beams always broke. Load-time and reaction-time curves as shown in Fig. 15 for the test of hard-grade beam R-126 tested pairwise with the steel beam at this energy level are typical. The initial load in this test of the hard-grade beam was 20 percent greater than the load required to break an intermediate-grade beam, when both were tested dynamically against a "stronger" beam. Two consecutive tests of *the same beam* are designated R-26 and R-126—the first at 7500 lb initial load, and the second at 9000 lb initial load.

When one beam was loaded dynamically to destruction, the concrete beam split along the reinforcing bar near the middle of the beam. In all tests, the concrete (6500 psi) started to disintegrate when the strain in the reinforcing steel reached about 3 percent.

There is no evidence in these tests to support the notion that a weaker beam protects a stronger (or stiffer) beam in a pairwise dynamic loading test.

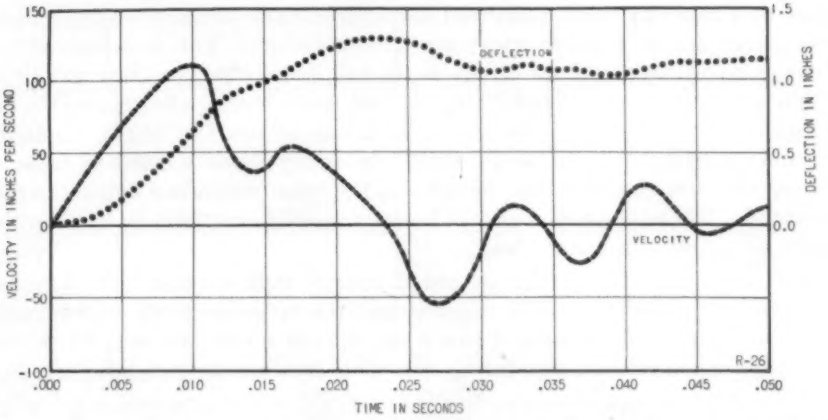

Fig. 14—Deflection-time and velocity-time curves for 7.5 kips suddenly applied to a hard-grade beam

There is ample evidence from all tests to show that the hard-grade beams outperformed the intermediate-grade beams consistently.

Dynamic behavior phases identified and discussed

When a motion picture which was taken at 1000 frames per sec is projected at 16 frames per sec the time scale is magnified about 60 times. Thus motion pictures provide an excellent means for a qualitative examination of dynamic tests. Frame-by-frame projections make accurate quantitative studies possible also.

Employing this technique in dynamic testing of concrete beams it was usually possible to identify three distinct phases of behavior. In the first phase, work

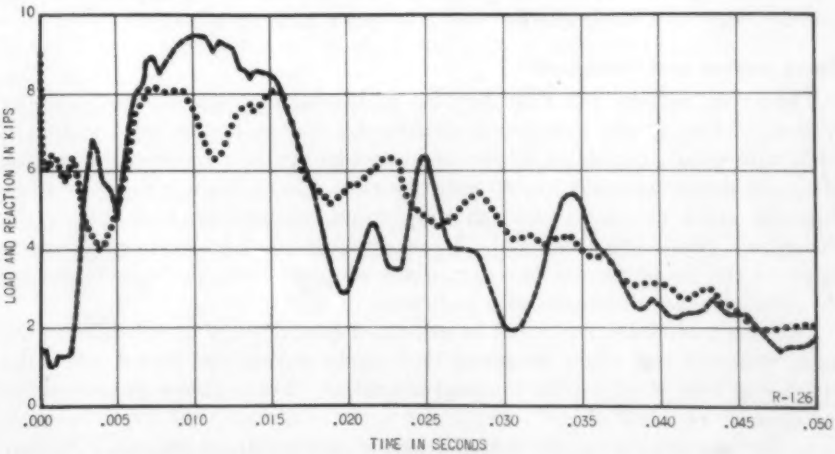

Fig. 15—Load-time and deflection-time curves for a 9.0-kip test

done on a beam by the applied load is converted into primarily elastic potential energy and to a lesser extent into kinetic energy. This is evidenced by large, continuous curvature of the beam and small but increasing velocity. There is no evidence of "plastic hinges" (indicating energy-absorbing inelastic strain in the steel), cracks, or impending failure in this first phase. Indeed, when tested in pairs, the beam which eventually failed seemed to behave during this first phase exactly the same as the beam which had ample reserve strength. This phenomenon cannot be satisfactorily explained by considering only the inertia of the materials.

For these tests, the first phase lasted roughly 0.01 sec and seemed to be independent of the load. This suggests that the materials of which the beam was constructed may be subject to a yield delay of about 0.01 sec. Certainly the stress-strain relationships for the materials involved are functions of time.

The first phase can be easily identified in Fig. 7 as the time period (roughly 0.01 sec) during which the deflections of the intermediate-grade beam and hard-grade beam are alike. During this same interval the velocity shown in Fig. 14 increases. The load is generally greater than the reaction as shown in Fig. 13 and 15.

The second phase occurs as the bulk of the elastic energy is released. During this phase, an increasing amount of energy is converted into kinetic energy of the two halves of the beam accelerating angularly. Throughout the second phase, a relatively steady amount of energy is irreversibly absorbed by inelastic yielding of the beam in a so-called "plastic hinge" developing under the point of load.

The third phase occurs as the kinetic energy which was stored during the first and second phases is released. During this phase the beam is brought to rest by the braking action of the plastic hinge as it yields inelastically. If the kinetic energy of the two halves of the beam exceeds the capacity of the plastic hinge to absorb energy, then the beam fails completely.

Crack pattern and "whiplash"

The crack pattern was examined by projecting the camera film frame by frame. When simply supported reinforced concrete beams were tested at high rates simulating blasts, all the usual diagonal tension cracks characteristic of simple beams statically loaded were observed. In addition, a single counter-diagonal crack extending through the normal compression zone during the "whiplash" phase was also noted. These cracks in what is *usually* compression concrete can be prevented by appropriate whiplash steel, perhaps improving the over-all blast resistance of a structure.

The origin of these cracks can be explained qualitatively by considering the load, moment, and shear diagrams that apply during the period when the reaction is zero shortly after the load is applied. Fig. 6 shows this condition for time $T = 0.002$ sec.

As the load builds up, the initial reaction quickly drops to zero. At that instant, all of the beam is in motion but not all of it is moving in the same

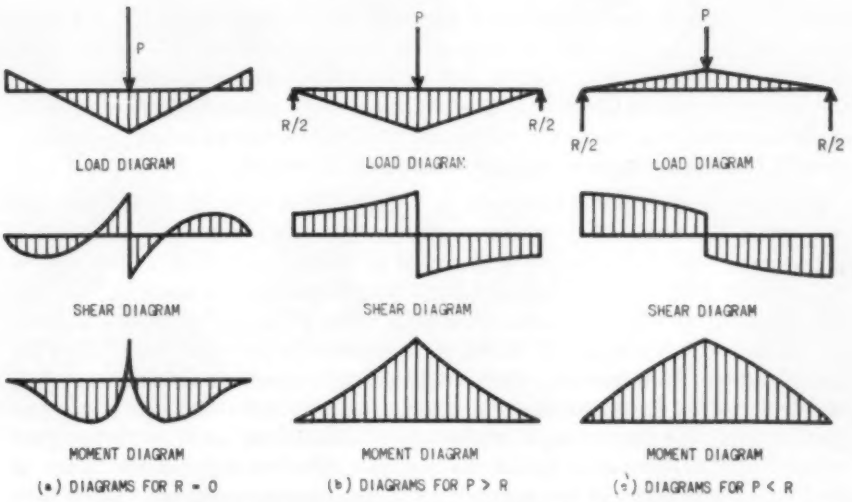

Fig. 16—Load, shear, and moment diagrams for a dynamic test

direction. The center of the beam is moving in the direction of the applied load while the ends of the beam are *backing off from their reaction support.* While the reaction is zero, the load, shear, and moment diagrams are somewhat as shown in Fig. 16(a). Here the effects of acceleration are treated as a distributed load in the same way the effect of the acceleration of gravity (weight) is treated as a distributed load under static conditions. During this period, the moment and shear in parts of the beam are opposite in sign from what they are normally; hence the diagonal tension cracks are reversed and appear on what is normally the compression side of the beam. If there is no steel—or not enough steel—in what is temporarily the tension side of the beam, cracks will form in the face of the beam which would be in compression under normal static loads.

The section where whiplash cracks first appear can be found analytically in terms of an implicit function which involves: (a) load as a function of time, (b) end conditions, (c) mass of the beam, (d) dimensions of the beam, (e) the "forward" (ordinary) yield moment, and (f) the "reverse" limit moment (based on uncracked section and steel in the compression zone). The solution of this implicit function requires more time than the result would seem to justify.

Shear distribution and bond failure

No evidence of bond failures was observed at the ends of any of the bars. Reinforcing bars with ASTM A 305 deformations appeared to offer sufficient bond resistance near the ends where the tension was low.

Bond failure observed near the center of the beams, extended well beyond the portion of the bar which exhibited permanent set and beyond the point

where it could be due to spalling of the mill scale. This might be accounted for by considering first that the bond *stress* due to dynamic load is not uniform along the beam as it is for a simple beam loaded with a concentrated static load, and second, by considering the bond *resistance* to be a function of the tension in the steel and the properties of the steel as well as the generally recognized criteria involving concrete strength and steel dimension.

To simulate the conditions found in dynamic tests by a static test it would be necessary, among other things, to obtain a moment and shear distribution in the static test beam similar to that due to the dynamic load. Only part of the dynamic load may be resisted by the beam's reaction at one time, the rest being resisted by a distributed inertial load. The shear and moment diagrams for the dynamic loading will therefore in general be different from those for static loading. For example, when a high-energy concentrated load is applied, the shear will be the same as if the applied load were divided into parts, one part resisted by the reaction, under static conditions, and the other part resisted as if it were distributed the way the effective mass of the beam is distributed lengthwise of the beam. The composite shear diagram taking both parts into account will be something like Fig. 16(b) if the load is greater than the reaction.

The moment diagram likewise can be thought of as a composite of a linear portion due to the reaction plus a nonlinear portion due to the beam's inertia. Fig. 16(b) suggests the idea.

Fig. 16(c) shows schematically, load, shear, and moment diagrams when the reaction is greater than the load.

Steel strain and energy absorption

Strain capacity and strain requirements for reinforcing bars are a concern to many investigators of reinforced concrete beams. The strain capacity is generally determined and reported on the basis of a standard gage length which includes the neck. Minimum requirements for percentage elongation have been conservatively established in standard specifications. Standardization of requirements in terms of minimum percentage elongation has, in some respects, arrested progress. On the one hand, there may be a tendency to place undue weight on percentage elongation data and lose sight of the fact that part of the elongation is due to the necking of the specimen. On the other hand, it is easy to assume that if a little ductility is good, a lot is better. This assumption is reasonable provided that the excess ductility serves some useful purpose, and that it is not obtained at the expense of some more desirable property.

Whatever ductility a reinforcing bar has in excess of about 3 percent elongation (in an 8-in. gage length) serves no useful purpose as beam or slab reinforcement since concrete starts to disintegrate when the strain in the steel reaches about 3 percent. This is far less than currently specified minimum elongation for even hard-grade steel reinforcing bars. Yet excess ductility

is generally obtained at the expense of tensile strength which contributes directly to the load capacity of the beam.

Strain distribution along representative full size samples of hard-grade and intermediate-grade bars, which were broken in static tension tests, is shown in Fig. 9 and 11. Strain as ordinate is based on a gage length of 2 in. and plotted with an abscissa at the center of the gage length. Similar plots are shown in Fig. 10 for intermediate-grade bars taken from typical 8.5-, 7.5- and 6.5-kip tests. The center of each gage length is shown as abscissa relative to the center of the beam. The residual strain was confined to a length of bar approximately equal to twice the depth of the beam. The area under these curves represents total strain. The product of the total strain and the average force causing the strain is equal to the energy absorbed by the bar in yielding.

It is clear from these tests and analyses that the hard-grade steel bars had an ample reserve of ductility plus a distinct advantage of higher yield and ultimate strength as compared with intermediate-grade or structural-grade bars as reinforcement for concrete beams subjected to destructive impulse loads. Neither in their laboratory tests nor in their review of voluminous literature (see bibliography at end of this paper) have the authors found experimental data to support the statement quoted (footnote p. 233) regarding reinforced concrete *beams* subjected to heavy, damaging blast forces.

SUMMARY AND CONCLUSIONS

1. Pairwise dynamic tests of two concrete beams that are alike except for grade of reinforcement offer a rational and consistent means of comparing resistances of structural elements to simulated blast loads.

2. The performance of a reinforced concrete beam under high rate dynamic loading is different from the corresponding static performance of a similar beam in that

a. The dynamic load capacity is greater.
b. The limiting dynamic strain is greater.
c. The dynamic reaction varies differently from the load (although the impulse integrals of load and reaction are equal from rest to rest, considering the system as a whole).
d. The dynamic stress distribution is different.

3. In all tests of beams that were alike except for grade of reinforcement, beams reinforced with hard-grade reinforcing steel outperformed beams reinforced with intermediate-grade or structural-grade steel in both static and dynamic elastic load capacity, and static and dynamic ultimate load capacity

4. The authors have found no test data to indicate that structural or intermediate-grade steel is better for reinforcement against damage due to heavy blast forces than hard-grade steel.

ACKNOWLEDGMENTS

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Modern concrete bridges

The Engineer (London), V. 203, No. 5281, Apr. 12, 1957, pp. 582-584

Reviewed by ARON L. MIRSKY

Illustrated report of lecture on "Modern Conceptions in the Planning of Bridges in Reinforced Concrete and in Prestressed Concrete," by Riccardo Morandi (London, Joint Committee on Structural Concrete, Mar. 19, 1957).

First part of lecture, on arches, stressed modern techniques which permit construction of long-span, small-rise structures with a considerable reduction in the portion of the carrying capacity devoted to dead load. Second part concerned beam bridges in both reinforced and prestressed concrete.

Construction

Japanese architectural techniques

Architectural Institute of Japan, Tokyo, 1957, 68 pp.

Compiled for the 70th anniversary of the institute, the book contains 16 papers by Japanese architects, divided into sections dealing with building materials, construction, structural design, thermal protection, fire protection, mechanical and electrical facilities, and one paper on the repair of ancient buildings to preserve cultural properties.

New quays and piers constructed in the port of Gothenburg (in Swedish)

ALLAN BERGFELT, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 155-180

Reviewed by MARGARET CORBIN

Describes two quays and two piers constructed during 1955 and 1956 in Gothenburg. A common feature of all these structures is the fact that they are founded on rock, either directly or by means of piles. This is a departure from the traditional Gothenburg type of quay, which comprises a concrete deck supported on cohesion piles. Prefabricated concrete parts were used in two of the structures.

Foundations of the Jochenstein dam power plant (Die Gruendung des Kraftwerkes der Staustufe Jochenstein)

RUDOLF KUHN, *Der Bauingenieur* (Berlin), V. 31, No. 6, June 1956, pp. 197-204

Reviewed by ARON L. MIRSKY

Site geology (including ground-water levels), form of power plant, loading, and hydraulic efficiency dictated design of turbine casing, intake, and outlet structures for this power plant on the Donau. (See also VDI *Zeitschrift*, Dec. 11, 1955, pp. 1265-1272; "Current Reviews," ACI JOURNAL, May 1956, *Proc.* V. 52, p. 1018.)

Construction of secondary treatment plant at the L. C. C.'s northern outfall

The Engineer (London), V. 203, No. 5282, Apr. 19, 1957, pp. 610-612

Reinforced concrete structures with a very smooth surface

Engineering (London), V. 183, No. 4753, Apr. 12, 1957, p. 478

Reviewed by ARON L. MIRSKY

Specifications for concrete for aeration chambers of London County Council's Northern Outfall Works required surfaces to be very smooth "to the satisfaction of the engineer." Result was to be obtained by using special care, lined forms, and "punning;" no rubbing or dressing down of the surfaces was permitted. Actual results, obtained by

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 3, Sept. 1957, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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lining forms with steel plate, using high (up to 0.5) water-cement ratios, and special care in placing and consolidating, were very good. Traveling forms were used. Total concrete volume amounted to some 70,000 cu yd.

Dams

Arch dams: Review of experience

ROBERT E. GLOVER, *Proceedings*, ASCE, V. 83, PO 2, Apr. 1957, pp. 1217-1 to 1217-48

AUTHOR'S SUMMARY

Presents records of the performance of some of the older arch dams to learn how well they had served the purposes of their designers. The records are presented in tabular form, supplemented by notes and pictures.

Concerning the construction of the new Cancano II dam at Fraeele (Vom Bau der neuen Staumauer Cancano II di Fraelle)

VOLKER WIGAND, *Der Bauingenieur* (Berlin), V. 31, No. 6, June 1956, pp. 213-215

Reviewed by ARON L. MIRSKY

Dam, in the Italian Alps, is of arch-gravity type of imposing dimensions: length at crown 350 m current construction (440 m future), maximum height above foundation 133 m (173 m future), volume 490,000 cu meters (1,076,000 cu m future), base thickness 25.9 m (future 39.9 m), crown thickness 5.4 m (5.5 m future). Reservoir created by new dam drowns out existing Cancano I dam.

Design

Cylindrical thin shell roofs of circular section (in French)

N. DEHOUSSE, *Bulletin*, CERES, Liège, V. 7, 1955, pp. 3-118

Reviewed by E. D. ROLIN

In 110 pages the author gives the full mathematical treatment to cylindrical shells on two supports with uniform load. Flügge's three linear partial differential equations are derived. These are solved simultaneously and by summing up the complementary function (flexural theory) and the particular function (membrane theory). Transverse loading and strains in the X, Y, and Z planes are taken as trigonometric functions, the first term of the Fourier series being considered only. Fifteen elaborate expressions are derived for direct, shear, bending, and torsional stresses in the X, Y, and Z planes as well as for the strains and rotation of the shell. Tables are furnished in terms of Jakobsen's parameters assuming Poisson's ratio of 0.2. An example is solved, incompletely, however, but it is not apparent how the complementary function varies with the external load.

Buckling of spherical shells by external pressure (Third report: Buckling mechanism of spherical shell segment)

MASUJI UEMURA, *Proceedings, Third Japan National Congress of Applied Mechanics* (Tokyo), 1953, pp. 113-118

Reviewed by GUNHARD-AESTIUS ORAVAS

Buckling and "durchschlag" ("canning") of a spherical shell cap is investigated by Tsien's principle of energy levels. For the most part it is a repetition of Tsien's work.

Fundamentals of statically indeterminate structures

C. L. SHERMER, Ronald Press Co., New York, 1957, 264 pp., $6.50

An up-to-date text and reference book on structural analysis. Modern and classical methods of analysis are included. Rather than presenting new or original material, the book collects methods of structural analysis in common use, presents a clear picture of the development from fundamental concepts of statically indeterminate structures, and includes examples of their application to a variety of problems.

Limit analysis and ultimate strength design are set forth in a simple manner as supplementary methods to the elastic analysis.

Design of elevated water tanks (Calcul des cuves des châteaux d'eau)

R. GAUTHRON, Editions Eyrolles, Paris, 1957, 218 pp., 4040 F

A study of elevated water storage reservoirs in reinforced concrete. The first part deals with selection of an economical shape appropriate for the capacity and head required and esthetically pleasing. Discusses general forms that have been used and those which may be utilized. The second portion gives examples and illustrates practical calculations necessary, including curves and data for design.

On some extensions of the theory of rigid-jointed folded-plate structures with prismatic, narrow individual plates (Ueber einige Erweiterungen der Theorie des steifknotigen Faltwerkes mit prismatischen, schmalen Einzelscheiben)

H. AICHINGER, *Der Bauingenieur* (Berlin), V. 31, No. 6, June 1956, pp. 216-221

Reviewed by ARON L. MIRSKY

Continuation of article in same journal, V. 30, No. 11, Nov. 1955, pp. 397-403 ("Current Reviews," ACI JOURNAL, Jan. 1957, *Proc.* V. 53, p. 712). Includes a section on T-beam or composite-type bridges treated as folded plates.

Discussion—Application of an electronic digital computer to some problems of structural analysis

The Structural Engineer (London), V. 34, No. 8, Apr. 1956, pp. 293-301

Reviewed by C. P. SIESS

Discussion of paper in *The Structural Engineer*, January 1956 (see "Current Reviews," ACI JOURNAL, Aug. 1956, *Proc.* V 53, p. 235). Horne described problem of buckling of triangular framework solved on computer. Wood suggested application to buckling of parallel plates (as in a steel column) and to analyses of slabs on beams.

Design and construction of a helical staircase

A. H. MATTOCK, *Concrete and Constructional Engineering* (London), V. 52, No. 3, Mar. 1957, pp. 99-105

Includes fairly complete theoretical analysis based on assumption that top and bottom supports are rigid vertically, that the vertical load on each is half the total weight of the staircase and applied loads, and that there is no restraint moment about horizontal axes at either the top or bottom supports.

Notes on the plastic analysis of concrete (Notes sur le calcul plastique du beton)

BERNARD LAMARRE, *L'Ingenieur* (Montreal), V. 42, No. 165, Spring 1956, pp. 14-19

Reviewed by ARON L. MIRSKY

Discusses distribution of the ultimate stresses in the compression zone of a reinforced concrete beam in simple flexure, for different concrete strengths and rates of loading.

Strength of a horizontal reservoir with equidistant supports

KOKI MIZOGUCHI, *Proceedings, Third Japan National Congress of Applied Mechanics* (Tokyo), 1953, pp. 47-50

Reviewed by GUNHARD-AESTIUS ORAVAS

Author solves the problem of a horizontal thin cylindrical tank supported by stiffening rings and loaded by hydrostatic pressure. The results in general are well known.

Practical calculation formulas and graphs of earthquake stresses in vertical hollow cylindrical shell

TADAAKI SAKAI, *Proceedings, Third Japan National Congress of Applied Mechanics* (Tokyo), 1953, pp. 123-126

Reviewed by GUNHARD-AESTIUS ORAVAS

A simplified shell theory is applied to earthquake analysis of thin cantilevered cylindrical shells. Graphs are given, which are applicable to cylinders of various proportions.

Concept confusion on the reinforced concrete sector? (Begriffsverwirrung auf dem Stahlbeton-Sektor?)

OTTO MULLER-MAIN, *Der Bauingenieur* (Berlin), V. 31, No. 9, Sept. 1956, pp. 340-341

Reviewed by ARON L. MIRSKY

Pointed comment on the argumentation pro and con the ultimate load method of design at the 1955 convention of the German Concrete Association (see "Current Reviews," ACI JOURNAL, Nov. 1956, *Proc.* V. 53, p. 560). Author is of opinion only stresses below the elastic limit (roughly, but not identically, equal to the proportional limit) have meaning in concrete. He is highly critical of attempts to design using factors of safety which give allowable (working) stresses in the "elasto-plastic" region, which he characterizes as a region of *overload*, not merely "higher loading." One reason stated is the irreversible damage to the concrete matrix in this region (shades of Homer Hadley!*). Another is the difference in effect of long-time and short-time loadings. Author is especially critical of the attempts to use ultimate load concepts in prestressed concrete design. He concludes with a discussion of the value of n, the modular ratio.

*"When Concrete Becomes Discrete," *Civil Engineering*, V. 20, No. 4, Apr. 1950, pp. 29-31.

Analysis of hipped plate structures by influence coefficients

B. N. THADANI, *Indian Concrete Journal* (Bombay), V. 31, No. 4, Apr. 1957, pp. 106-114

Presents a method for the analysis of hipped plate structures considering the effect of joint displacements. Develops theory for the method, and then illustrates its application by means of several examples.

Design of shell roofs. Part 3—North light shell

A. CHRONOWICZ, *Civil Engineering and Public Works Review* (London), V. 51, No. 605, Nov. 1956, pp. 1242-1243

Presents an example of the analysis required for a north light shell based on simplifying assumptions which are said to result in negligible discrepancies.

Designing concrete members by assuming the neutral axis

CHARLES E. SHARP, *Consulting Engineer*, V. 9, No. 4, Apr. 1957, pp. 64-67

Reviewed by ARON L. MIRSKY

Trial-and-error solution, especially useful for members of irregular cross section or members with multiple layers of steel. Application of method to such members in pure bending or in bending plus direct stress is illustrated by several worked examples.

(A modification of this method is described by Harold J. Jobse in a letter to the editor in title source, June 1957, pp. 20, 22.)

Distribution method for circular cylindrical shells

J. C. DE C. HENDERSON and E. KEMP, *The Engineer* (London), V. 203, No. 5283, Apr. 26, 1957, pp. 634-638

Reviewed by ARON L. MIRSKY

Describes and explains an edge-disturbance distribution method devised by R. S. Jenkins (*Theory and Design of Cylindrical Shell Structures*, privately published, 1947) for single or multiple shells. Method, intended for the analysis of reinforced concrete shells, assumes an elastic homogeneous cross section; in addition, it assumes the "other" edge to be infinitely distant, instead of fixed as in the moment-distribution method. Prospective readers of this paper should be familiar with the elastic theory of the cylindrical shell and with matrices, which are here used as linear operators. (Tabulated values of two of these matrices are available, for use in calculation.)

Degree of redundancy of plane frameworks

J. A. L. MATHESON, *Civil Engineering and Public Works Review* (London), V. 52, No. 612, June 1957, pp. 655-656

An attempt to present an unambiguous formula for computing the degree of redundancy.

Practical stress calculation table of cylindrical tanks with linearly varying wall thickness

TADAAKI SAKAI, *Proceedings, Fourth Japan National Congress of Applied Mechanics* (Tokyo), 1954, pp. 163-168

Reviewed by GUNHARD-AESTIUS ORAVAS

Author gives tabulated functions for easy calculation of cylindrical tanks with linearly varying wall thickness. The tank has a free edge on the top and fixed edge in the bottom. Loading is hydrostatic. Meissner's solution is used in the theoretical development.

Approximation of the bending theory of a shell

YOSHITSURA YOKOWO and OSAMU MATSUOKA, *Proceedings, Third Japan National Congress of Applied Mechanics* (Tokyo), 1953, pp. 43-45

Reviewed by GUNHARD-AESTIUS ORAVAS

Fundamental approximate shell equations are derived in this paper for the analysis of thin shells. Tensorial notation has been used throughout and the dependent variable has been introduced in a complex form comprised of stress function and normal displacement function. Reviewer feels that from engineer's standpoint the authors' presentation of basic thin shell equations is best suited for practical calculations. A lot of material in this paper overlaps existing work on the subject.

Approximation of the bending theory of a spherical shell

YOSHITSURA YOKOWO and OSAMU MATSUOKA, *Proceedings, Fourth Japan National Congress of Applied Mechanics* (Tokyo), 1954, pp. 177-181

Reviewed by GUNHARD-AESTIUS ORAVAS

An asymptotic solution has been given in this paper for spherical shells. Basic equations are developed from generalized tensorial thin shell equations set up by the same authors in a previous paper (see review above). Interesting comparisons are made with the solutions of existing developments on the antisymmetrically stressed shell.

Dimensioning of plate girders (Dimensionierung von Biegetraegern)

KARL MINK, *Der Bauingenieur* (Berlin), V. 31, No. 5, May 1956, pp. 172-174

Reviewed by ARON L. MIRSKY

Develops simple expressions for direct design of composite girders and of the steel portion alone. A numerical example involving a box-type section is included.

Materials

Some observations about the study of building materials (Algunas observaciones sobre el tema del estudio de los materiales de construcción)

MARCEL LEPINGLE, *Bulletin* No. 179, Instituto Técnico de la Construcción y del Cemento, Madrid, 1956, 19 pp.

Reviewed by ROGER DIAZ DE COSSIO

A discussion of the different types of cement existing in the market, and the tendency of going toward higher and higher resistances. Other types of building materials are also discussed. Author advocates a thorough investigation of building materials made by an impartial organization.

Complex hardening accelerators of cements (in Russian)

Y. M. BUTT and G. S. ROYAK, *Zhurnal Prikladnoi Khimii*, V. 29, No. 1, 1956, pp. 7-10

CERAMIC ABSTRACTS June 1957 (Kamich)

Complex additions consisting of sodium sulfate and calcium chloride lead to considerable acceleration of hardening during all periods and are especially effective for slag-portland cements under conditions of natural hardening and hydrothermal treatment.

Optimum granulometric composition of concrete aggregates (Der Bestwert der Kornzusammensetzung von Betonzuschlagstoffen)

FRITZ SCHWANDA, *Der Bauingenieur* (Berlin), V. 31, No. 2, Feb. 1956, pp. 41-46

AUTHOR'S SUMMARY

Treatise starts from presumption that the cement paste has to cover every single aggregate particle and to fill all interstices between them. A formula and a graphical method are derived for the calculation of the void space of mixes of two size groups. Two different formulas are valid, depending on whether few coarse lumps are embedded between a mass of fines, or whether the fine particles fill only small corners of the interstices between the coarse ones.

Cost and yield of concrete mixes for various particle gradations are studied with aid of triangular-coordinate plots of combinations of three sizes (fine, medium, coarse) of aggregates. This method has successfully been applied on several large jobs in Austria. As the gradation of the coarser size groups exercises a small influence on the cement demand of a mix, it is in many practical cases sufficient to find the proportions of the finest size group, the second finest, and the rest.

(See also "Graphical Determination of the Grading of Combined Aggregates," *Concrete and Constructional Engineering* [London], V. 52, No. 5, May 1957, pp. 187-190.)

Special type of slide rule to calculate the potential composition of portland cement (in Spanish)

ANTONIO SARABIA, *Bulletin* No. 177, Instituto Técnico de la Construcción y del Cemento, Madrid, pp. 5-14

Reviewed by CELSO A. CARBONELL

Describes a special type of slide rule to calculate the potential composition of portland cement. It also serves to work out some coefficients and values important in the chemical and technological applications of cement.

The author explains the basis of the design of the slide rule, as well as its actual construction, and mode of use. The values whose calculation can be effected with its aid are: potential composition, calcimetry, magnesia and lime content, and lime saturation index. Actual numerical examples are cited.

Use of high-tensile steel reinforcement

K. HAJNAL-KONYI, *Concrete and Constructional Engineering* (London), V. 52, No. 6, June 1957, pp. 197-209

Reviews the development of high-tensile steel as reinforcement for concrete and its use under various standards, particularly in England but also in the United States and European countries. Notes that the United States is an "extreme case" of conservatism whereas Austria is the opposite as far as utilizing high working stresses for high-tensile steel reinforcement. This article is the first of a series in which it is proposed to discuss theoretical and practical aspects of the applications of high-tensile steel as concrete reinforcement.

Fly ash concrete for Shippingport atomic power station

ROBERT J. MCALLISTER, *Proceedings*, ASCE, V. 83, PO 2, Apr. 1957, pp. 1215-1 to 1215-14

AUTHOR'S SUMMARY

About 15,000 cu yd of fly ash concrete using Type IS portland blast furnace slag cement have been placed at the Shippingport atomic power station. Paper deals with the solution of various problems, with particular emphasis on the development of a mix designed to attain a compressive strength of 3000 psi at 28 days.

Cements other than ordinary portland cements

Advisory Leaflet No. 39, Ministry of Works, H. M. Stationery Office, London, 4d; reviewed in *Cement, Lime & Gravel*, V. 31, No. 8, 1957, p. 380

CERAMIC ABSTRACTS June 1957

The cements described are rapid- and extra rapid-hardening portland, water-repellent portland, waterproof portland, sulfate-resisting, low-heat portland, portland-blast furnace slag, white and colored, super-sulfated, high-alumina, and masonry cements. The advantages and limitations of each are discussed, and the properties, uses, and trade names are given.

Apparent density of portland cement clinker (in Spanish)

ALBERTO VIRELLA BLODA, *Bulletin* No. 168, Instituto Técnico de la Construcción y del Cemento, Madrid, 1955, pp. 3-29

Reviewed by CELSO A. CARBONELL

This work contains the following: concept of real and apparent density; variations of this density in materials which are calcined to obtain portland cement clinker; ease in determining the apparent density; apparent density as a method useful in measuring the degree of burning of clinker. Also covered are the human factor, amount of heavy clinker, and proportioning of clinker as sources of error in clinker burning.

Automatic methods as means of reducing this error are treated, as well as expansion and free lime in clinker varying with apparent density, and their influence in the variations of chemical composition, especially the saturation of lime and the content of ferrous oxide.

Optimum density for best grinding on industrial and laboratory scale is shown. The same density gives highest strength values.

Relation between fuel consumption and apparent density of clinker—a good clinker may be more expensive in fuel, but there are compensations in grinding energy and in final quality of the manufactured product.

New binding material—Gypsum-slag cement (in Polish)

S. PIECZARA, *Cement-Wapno-Gips* (Stalinogród), No. 11, 1955, pp. 244-247

POLISH TECHNICAL ABSTRACTS No. 3 (23) 1956

The gypsum-slag cement is produced by grinding a mixture consisting of 85 percent granulated blast furnace slag and 15 percent gypsum. Cement clinker is added as an activator in quantities from 2 to 3 percent. Blast furnace slag containing less than 13 percent of Al_2O_3, is unfit for use in the production of cement. In bending strength it surpasses, and in compressive strength is equal to portland cement.

Tests of the adhesion of plaster to concrete

J. F. RYDER, *Chemistry and Industry* (London), No. 6, 1957, p. 169

CERAMIC ABSTRACTS June 1957

Describes two methods; the first measures force needed to pull a disk of plaster from a concrete surface and can be used for testing plaster work in buildings. The second method compresses the concrete to simulate the effects of drying shrinkage and differential thermal expansion and measures the strain at which failure in adhesion occurs.

Refractory materials for the cement industry (Materiales refactarios para la industria del cemento)

F. SORIA SANTAMARIA; *Bulletin* No. 165, Instituto Técnico de la Construcción y del Cemento, Madrid, 1955, pp. 3-51

Reviewed by CELSO A. CARBONELL

A survey of one of the most important and complicated problems inherent in cement kilns, and generally in installations working at high temperatures. The first part deals with raw materials, with special reference

to the types and properties of the materials in most common use, namely: alumina, magnesium, chrome products, and clinker.

The two basic types of kilns are explained, and the functional requirements of each zone of the kiln are analyzed. Construction and design of the lining types of brick, constructive methods, dilatation joints and thermal insulation are treated at length.

The importance of obtaining a stable lining, the system of firing and starting the kiln, and its possible breakdowns are also studied.

To consider the process of calcination a brief description of the theory of phases is given, since this is the basis for the study of the formation of a protective crust over the lining. This is followed by considering the related destructive phenomena of the lining in their various aspects: mechanical, thermal, and chemical.

Last part is devoted to the possibility of standardizing and defining the qualities of the materials employed in the cement industry.

Pavements

Standard specifications for construction of roads and bridges on federal highway projects

Bureau of Public Roads. Edition: FP-57, Jan. 1957, 364 pp., $2 (available from Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C.)

Book was issued primarily for use on federal highway projects. It contains specifications for those items of work, materials, and construction methods that are generally applicable to direct federal highway contracts, but it is adaptable for use by other highway agencies. Portions on concrete resemble the 1951 ACI specifications.

Precast Concrete

Prefabricated garages from Denmark

Prefabrication (London), V. 4, No. 44, June 1957, pp. 364-365

Describes two types of prefabricated concrete residential garages. One for small cars is prefabricated and delivered in one piece. The second model for larger cars is assembled at the site of prefabricated units with galvanized bolts.

Methods of manufacturing large slabs (in Polish)

F. Libiszowski, *Przeglad Techniczny* (Warsaw), No. 2, 1955, pp. 46-50

Polish Technical Abstracts No. 3 (23) 1956

For the industrialization of building methods it is necessary to manufacture large slabs and panels on a considerable scale. Three technological processes are possible: continuous line production, continuous stand production, and production on stands. The three methods are described in detail.

New technique for Turin factory

Pier Luigi Nervi, *Prefabrication* (London), V. 4, No. 43, May 1957, pp. 304-309

Describes precasting and erection of a three-story factory building. Precast girders weighing up to 14 tons supported precast floor and roof slabs. The slabs were cast in place on reinforced concrete pan forms supported on adjustable tubular scaffolding. The use of reinforced concrete forms, carefully finished, eliminated need for all other finishing material such as plaster.

Precast panels as formwork for a gravity dam

J. S. Ellis, *The Engineering Journal* (Montreal), V. 40, No. 1, Jan. 1957, pp. 24-26

Reviewed by Aron L. Mirsky

Brief article describing precast concrete panels used as upstream and downstream formwork for the Cluanie Dam in Scotland (see also *The Engineer*, Oct. 5, 12, 19, 26, Nov. 2, 9, 1956; "Current Reviews," ACI Journal, Apr. 1957, *Proc.* V. 53, p. 1016). Advantages of these panels are listed.

Corrosion of cement-asbestos and concrete pipe in some New Zealand soils

H. R. Penhale, *New Zealand Journal of Science and Technology* (Wellington), V. 38B, No. 3, 1956, pp. 257-278

Ceramic Abstracts June 1957 (Williamson)

Cement-asbestos and concrete pipe were exposed to soils for up to 10 years in five localities. Corrosion was negligible in well drained soils and slight in moist highly leached soils of high total acidity but without free ground water. It was severe where ground water rich in carbon dioxide flowed past the pipe.

Information concerning the design for the Warsaw experimental large-block building (in Polish)

Z. Kletff and B. Babicki, *Inzynieria i Budownictwo* (Warsaw), No. 7, 1955, pp. 233-236
Polish Technical Abstracts No. 4 (24) 1956

A design for a large block of flats, approximately 99 percent prefabricated. Vertical forces from the ceilings are borne exclusively by the longitudinal walls of the building, while the horizontal forces acting on the exterior longitudinal walls are directed, by means of horizontal diaphragm-ceilings, to the main transverse walls. The construction called for about 60 different types of prefabricated sections. Slag concrete was used.

Prestressed Concrete

Flexural resistance of prestressed simply supported concrete beams within the elastic range

H. W. Britton, *Journal*, Institution of Engineers, Australia (Sydney), V. 28, No. 1-2, Jan-Feb., 1956, pp. 39-46
Applied Mechanics Reviews Sept. 1956 (Borg)

Using the well known approximate formulas of prestressed concrete beam analysis, author develops a cut-and-try design procedure for certain assumed shapes. The method basically satisfies four required stress conditions by setting up four simultaneous equations in terms of arbitrary "shape factors." The equations, when solved, lead to two separate cases, "short spans" and "long spans," depending upon steel eccentricity. Design tables are included and two illustrative problems are solved.

Experimental investigation of the mechanical characteristics of the prestressing cables used for the reconstruction of the Villeneuve-St. George bridge over the Seine (in French)

Michel Bonnet, *Annales des Ponts et Chaussees* (Paris), V. 126, No. 4, July-Aug. 1956, pp. 463-526
Reviewed by Aron L. Mirsky

To aid in deciding tensioning device to be used in prestressing the cables for this bridge (for general description, see the *Annales*, Jan.-Feb. 1953, pp. 9-48; "Current Reviews," ACI Journal, June 1953, *Proc.* V. 49, p. 978), an extensive study was made of the mechanical properties of the cables and anchorages. Program involved factory tests of the component wires, laboratory research on their elasticity and breaking loads, site testing of the apparent modulus of elasticity of the cables, and tests of various types of anchorage.

Paper also reports (pp. 512-515) on the observed behavior of the cables in the completed structure. It was necessary to adjust the cable tensions several times during the first few years after completion. The prestressing system appears to have taken about 3 years to become stabilized; after that period cable tension remained appreciably constant.

For a similar investigation prior to specifying the concrete to be used in this structure, see the *Annales*, Mar.-Apr. and May-June 1953, pp. 185-228 and 287-314; "Current Reviews," ACI Journal, Nov. 1954, *Proc.* V. 51, p. 301.

Long-span prestressed concrete bridges constructed by the Freyssinet system

Yves Guyon, *Proceedings*, Institution of Civil Engineers, London, V. 7 (Session 1956-57), May 1957, pp. 110-168 (discussion, pp. 169-179)
Reviewed by Aron L. Mirsky

An excellent review of some of the best current practices in constructing long-span prestressed concrete bridges. This is not a textbook for the novice, nor is it a complete survey of the field; but it does cover a multitude of topics, and the experienced designer will gain much.

Subjects include: choice of bridge type (five types are critically evaluated, and the cost advantages of prestressed concrete are noted); preliminary design of prestressed bridges; design of substructures; concrete hinges and joints; and methods of construction (including the adjustment of reaction values). All points made are illustrated by reference to bridges both built and in the design state, all from the author's personal experience, including a number of less well-known structures. The discussion, while brief, brings up some interesting topics (author's closure appeared in the June 1957 issue, pp. 425-432).

A brief summary of this paper was published in *Engineering* (London), V. 183, No. 4747, Mar. 1, 1957, p. 271, under the title "Structural Engineering on the Continent."

Overload capacity—Prestressed compared with reinforced concrete

T. J. Bezouska, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 172-175

Compares overload capacities of simple span bridge girders in both prestressed and reinforced concrete as predicted by the accepted methods of ultimate strength design. Concludes that the overload safety factors based on total load and also based on live load are similar for both prestressed and reinforced concrete bridges. (This conclusion is to be expected since it was reached by working backward from the same theory used for proportioning the prestressed concrete bridges originally.)

Properties of Concrete

Note on the "Ocratation" of concrete

Trend in Engineering, V. 8, No. 3, July 1956, p. 15 (Translated from *Annales de L'Institut Technique due Batiment et des Travaux Publics* [Paris], Mar.-Apr. 1956, pp. 349-350.)

Highway Research Abstracts Jan. 1957

The process of "ocratation" of concrete was accidentally discovered by the Netherlands firm, Ocriet. It consists of treating concrete with silicon tetrafluoride (SiF_4) under pressure of around 4 atmospheres.

In its chief chemical reaction this process is similar to treatment by fluo-silicates, but its efficacy is much greater because the reactions can affect relatively important depths.

The chief reaction is: $2Ca(OH)_2 + SiF_4$ $CaF_2 + SiO_2nH_2O$.

Free lime, which is the weak point of all mixtures with a cement base, is used with a view toward (increasing) chemical resistance, and a gel of silica is freed which partially closes the pores of the concrete and increases impermeability. German tests of the process indicated 50 to 100 percent increase in compressive strength of concretes so treated. Resistance to wear was increased by about 50 percent; bond to reinforcement was increased by 30 percent.

AE-55 indicator used for determining air content of concrete

W. E. Grieb, *Public Roads*, V. 29, No. 9, Aug. 1957, pp. 206-207, 220

Reports field research to calibrate and determine limits of accuracy of the Chace pocket-sized air meter. The Chace air meter was calibrated against a standard sized pressure meter and the gravimetric method. It was concluded that the pocket-sized meter is not suitable as a replacement of the pressure or gravimetric methods for the determination of the air content of concrete, but is useful as a supplementary test for field inspection, and particularly convenient because of its small size and speedy operation in the field.

Corrosion of concrete (in Swedish)

Tenho Sneck, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 117-127

Reviewed by Margaret Corbin

Deterioration of concrete may be caused by physical and/or chemical factors. Corrosion of concrete is dependent on the properties of the hardened cement, the quality of the concrete and the aggressiveness of the surrounding medium. The effect of carbon dioxide and sulfur dioxide in the atmosphere is briefly discussed. The properties of very pure, soft and hard waters, waters with a low pH value, and the action of waters containing aggressive carbon dioxide are reviewed. The effects of humic compounds, magnesium salts, and sulfates or sulfides are also mentioned. Factors governing underground corrosion of concrete are discussed with particular attention to acidity of the soil, exchangeable hydrogen, humic compounds, and sulfur and magnesium salts. Some dangerous chemicals and corrosive environments are reviewed.

Analysis of stress distribution in indirect tensile tests on concrete cylinders

N. S. Bawa, *The Indian Concrete Journal* (Bombay), V. 31, No. 3, Mar. 1957, pp. 90-92

A theoretical analysis of the stress distribution in concrete cylinders used in the indirect tensile test, apparently based upon the elastic theory.

Concludes that "the computation of the tensile strength cannot be done by any simple formula," that the test is "unreliable for comparing different concretes," and furthermore, that the "proposed test bears no resemblance with the type of loading actually found under field conditions."

In summarizing the "advantages" claimed for the test, some of the most significant ones are conveniently overlooked.

Watertight concrete (Waterdicht Beton)

R. A. J. Bosschart, *Cement* (Amsterdam), V. 8, No. 21-22, Oct. 1956, pp. 527-528
Reviewed by John W. T. Van Erp

Watertightness may be important not only to prevent loss of liquid but also to reduce corrosive action of those liquids penetrating into the concrete. With any mix it is essential to keep the pores which are permeable to a minimum. Cement needs 1.4-1.9 gal. of water for the chemical hardening process; the rest will evaporate and leave pores, which are not harmful as long as they do not form interconnected channels that will allow the passage of water. To this purpose air-entraining admixtures are useful or even a certain amount of fines.

Structural Research

Experimental research on intermediate shells

Wataru Kato, *Proceedings, Third Japan National Congress of Applied Mechanics* (Tokyo), 1953, pp. 39-41
Reviewed by Gunhard-Aestius Oravas

A mortar model of a short, simply supported cylindrical thin shell is tested under uniform load. Stress and strain patterns are given. Conclusion: no membrane state of stress!

Plain reinforcing bars with end hooks versus deformed bars with widely spaced corrugations (in Swedish)

Lars Holmgren, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 209-220
Reviewed by Margaret Corbin.

Describes laboratory investigation, a comparative study of reinforcing bars of the steel quality St44 used as tension reinforcement in concrete beams. Bars were anchored either by means of end hooks or by widely spaced corrugations with an area of 0.23 d^2 and a spacing of 2.0 d. This corrugated steel is tentatively called Ks 26. The test bars had a diameter of 16 mm. The mean yield point of the smooth bars was 2.920 kg per sq cm and that of the corrugated bars 3.370 kg per sq cm. Tests were performed on 84 concrete beams of which 42 were reinforced with corrugated bars and 42 with smooth bars provided with end hooks. The beams were tested in flexural compression under two equal concentrated loads. The distance from the point of load application to the support was equal to the anchorage length; bond of corrugated bars was excellent. The calculated maximum bond stress amounts to 36 kg per sq cm, and at this stress no bond failure occurred. Thus, it ought to be possible to replace the smooth steel St 44 by the corrugated steel Ks 26. Crack spacing of beams reinforced with corrugated bars amounts to about 90 percent of that of beams with smooth bars.

Continuity in precast reinforced concrete

K. K. Banerjee, *Journal of Technology*, Bengal Engineering College (India), V. 1, No. 2, Dec. 1956, pp. 123-145

Efficiencies of some of the common types of joints were determined from experiments conducted on two model slabs, constructed of precast reinforced concrete units with both longitudinal and transverse joints. The variables in the tests were, span: width ratio of slab, types of transverse joint, and types of longitudinal joint.

Ultimate moment and yield point (in Dutch)

P. J. van Tussenbroek, *Cement* (Amsterdam), V. 8, No. 15-16, Apr. 1956, pp. 369-370
Reviewed by John W. T. Van Erp

Describes load tests on floors which showed a larger ultimate moment than could be expected from internal moment of steel at yield stress times leverage arm of plastically deformed concrete. Various reasons for this are analyzed.

Stress field of unsymmetrically loaded reinforced concrete wall with door opening (Ueber das Kraftfeld einer Stahlbetonwand mit einer Tueroeffnung unter unsymmetrisch wirkender Einzellast)

Ernst Zellerer and Hanns Theil, *Beton- und Stahlbetonbau* (Berlin), V. 51, No. 12, Dec. 1956, pp. 267-274
Reviewed by Valdis Lapsins

Study made with a reinforced concrete beam with a height to length ratio of 1:3.47 to determine stress concentration around beam opening and throughout the beam by comparing results of photoelastic method, model study, and theoretical knowledge.

Long-time deformation of reinforced concrete beams (in Swedish)

LENNART BACKMARK and ARNE I. JOHNSON, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 181-190
Reviewed by MARGARET CORBIN

Early removal of forms is generally desirable today. An uncertain factor is the effect of early form removal on long-time deformation of floor slabs. Tests showed that, for a given type of concrete, the time of load application and the method of curing do not produce any appreciable effects on long-time deformations for the period covered by the tests—2¼ years. The type of concrete has a considerable effect on long-time deformations. Deflections are substantially increased when low-quality concrete is used. Deformations causing cracks in partitions occur after erection of partitions. If forms are stripped early, a large part of the total long-time deformation in the slab takes place before partitions are erected. Consequently, early stripping of forms is frequently desirable. Variations in humidity caused a permanent increase in deflections ranging from 5 to 6 percent. This result may be viewed as a possible explanation to the fact that floor slabs above rooms in which air moisture varies within wide limits, such as some boiler rooms, often undergo large long-time deformations.

General

American civil engineering practice

ROBERT W. ABBETT, editor, John Wiley & Sons, Inc., New York; V. 1, 1956, 1026 pp., $15; V. 2, 1956, 956 pp., $15; V. 3, 1957, 1200 pp., $25

In scope, the series is intended to cover the recognized technical and professional specialties. Supplemented by *Handbook of Engineering Fundamentals* by Eshbach, and *Handbook of Engineering Materials* by Miner and Seastone, it provides a complete reference library covering all of the usual problems likely to arise in civil engineering. *American Civil Engineering Practice* is subdivided into 34 sections and published in three volumes. Each volume covers a broad field of civil engineering and is a complete entity. V. 1 contains metropolitan and community planning, surveying, traffic, highways, airports, railroads, soil mechanics and site planning, foundations, earthwork, tunnels, and mathematical tables. V. 2 is devoted to the general fields of hydraulics, sanitary, and harbor engineering; V. 3 discusses masonry, reinforced concrete, steel, and timber structures. Each volume is separately indexed. Each section has been written by a recognized authority in its field.

Large unobstructed floor structures (Massivhallenbauten)

RUDOLF OHLIG, *Der Bauingenieur* (Berlin), V. 31, No. 2, Feb. 1956, pp. 55-61
Reviewed by ARON L. MIRSKY

Descriptions of various structures constructed in 1953 and 1954. Structural systems used to permit large column-free floors include: long-span prestressed concrete girders, folded plate roofs, and multiple prestressed cylindrical shells.

Investigation of the improved Carlson stress meter

E. E. MCCOY and B. MATHER, *Technical Report* No. 6-454, Waterways Experiment Station, May 1957, 50 pp.

Presents detailed results and analyses from an investigation to evaluate the suitability of the improved Carlson stress meter for use in 6-in. aggregate concrete. The conclusions are that the stress meters are sufficiently sensitive and independent of the effect of extraneous length changes for practical applications in both 1½-in. aggregate concrete and 6-in. aggregate concrete.

Guide to atomic energy literature for the civilian application program

United States Atomic Energy Commission, Technical Information Service, TID-4575, 1957, 74 pp. (Available without charge from AEC Technical Information Service Extension, P. O. Box 1001, Oak Ridge, Tenn.)

First part describes informational materials and services provided by the Atomic Energy Commission—research and development reports, books and special compilations of technical data, bibliographies, abstract journals and announcement bulletins, library catalog cards, engineering material lists, and a literature search service. Parts II and III explain how to find and use this resource material.

Appendices list volumes in the National Nuclear Energy Series; special publications and bibliographies; categories of restricted data available to access permit holders; AEC depository libraries; consolidated listings of

unclassified and confidential reports; letter and number codes used in identifying reports; and explain the indexing method used in abstract journals.

Engineering structural failures

ROLT HAMMOND, Philosophical Library, New York, 1957, 224 pp., $12

Reviewed by JACOB FELD

The scant literature on the subject is considerably enriched by this work. Except for comparatively few technical articles on the cause of failures, as compared with the large number of descriptions of successes in engineering work, there have previously appeared only a short book, *La Pathologie du Beton Arme* by Henry Lossier (1952), and a translation of *Landslides*, the 1854 work of Alexandre Colin by W. R. Schriever (1956). The reviewer has recently published several lectures on the causes and lessons of structural failures and in so doing has been the subject of some criticism in suggesting an open forum so that all may learn from mistakes made. It is evident that the author is similarly restrained, and much available information is not included in his book.

Eight general classes of engineering structures are covered: earthworks, dams, maritime structures, buildings, bridges, underground structures, vibration problems, and welded structures, followed by a chapter on lessons of failures. Had the author not deviated from his topic by including several interesting and informative discussions not pertinent to the subject, more space would have been available for the many recorded failures in each of the classes covered, thereby showing the generally consistent patterns which seem to be conducive to "incidents" in engineering structures.

The random samples of earthwork failures described are a weak analysis of causes; much more authentic data is available in numerous reports on soil mechanics issued since 1930. The author's grasp of this subject is inferior to his understanding of construction techniques. Only a few spectacular dam collapses are described; yet the lesson of the Fort Peck failure is omitted. The discussion on arch dam design seems out of place. Similarly, almost the entire chapter on maritime structures gives us good introduction to design and forces acting and a complete theory of wave action, but very little of the many sea wall and groin failures.

There are descriptions of five building failures in the United States from 1901 to 1924. There are available many examples of more recent construction design and practice which would be of greater interest, and strangely enough there are given no examples of building failures in the country of this book's origin. The description of the troubles with expansive clays (the author calls them "shrinkable clays") is not a complete story; much better information is found in the soil reports from Texas, Canada and Southern Rhodesia.

The chapter on bridges goes into great detail on the Quebec and Tacoma Bridge collapses, but misses the opportunity to teach the lessons of distress in small bridge designs and of the typical erection failures. The subject of underground structure failures includes a dissertation on the theme that soil is not a static material and that tests of pressures are possible, but only one shaft and two tunnel failures are described without correlation to the theoretical discussion. Some mention should have been made of small tunnels and conduit failures and of the recent Wilson Tunnel incident in Hawaii.

Vibration problems are treated in a form suitable to an engineering handbook by description of vibrograph recording of earthquake, blasting and machinery effects, very well presented, but the description of insufficiency in actual designs is not covered.

Almost the entire chapter on welded structures is taken by discussion of steel metallurgy, brittle fracture and metals of low fatigue strength. The description of welded steel ship failures is good as is also the author's vindication of the preference of welded over riveted ship design.

The concluding chapter has a misleading heading, since no lessons of failures are given. There is instead a description of modern instruments for stress determination and of full scale tests of structures. The author has done service in opening up the literature on the subject but has not provided good coverage and missed the opportunity to fill the gap in engineering knowledge. It has been said that doctors bury their mistakes and architects plant ivy. Engineers write reports and then bury the lessons to be found therein.

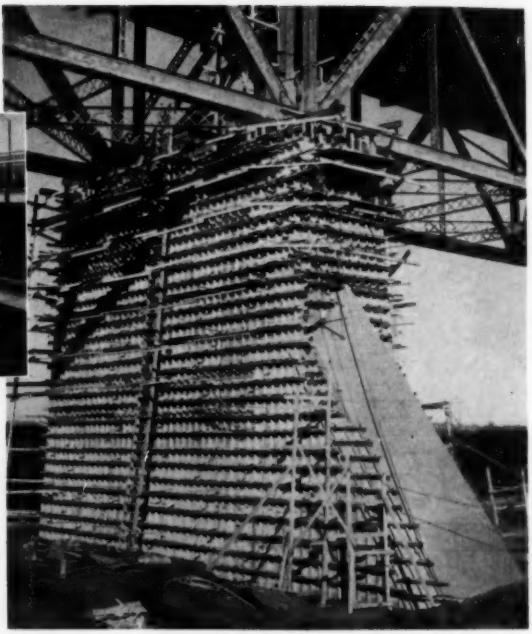
ACI News Letter

Vol. 29, No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1957

Seattle, ACI regional meeting site

Consulting Engineer: Dr. P. L. Pratley
General Contractor: Dominion Bridge Co. Ltd.
Substructure Contractor: Atlas Construction Co. Ltd.
Concrete Contractor: Intrusion-Prepakt, Ltd.
Photos: Courtesy of Roads and Engineering Construction

Piers in foreground have been enlarged with Prepakt concrete, completing the initial stage of a complex bridge reconstruction project.



Prepakt concrete was used to enlarge eleven piers of the Jacques Cartier Bridge, demonstrating the advantages of I-P methods — high strength, ease of placement and flexibility of operation.

Prepakt speeds reconstruction of St. Lawrence Seaway bridge

Intrusion-Prepakt, Ltd. recently played a vital role in the reconstruction of the Jacques Cartier Bridge at Montreal. The highly complex project was undertaken to provide minimum clearance of 120 feet for ocean-going vessels soon to use the St. Lawrence Seaway canal. Fourteen spans of the bridge, a total of 2625 feet in length, will be jacked up to new elevations on raised piers, a new end span will be inserted and the center span replaced—all without disturbing traffic.

Recognizing the advantages of Prepakt, Seaway authorities specified its use for increasing the thickness of eleven piers an average of 1½ feet on all surfaces. Selected coarse aggregate was placed in the forms and consolidated with Intrusion grout to form high-strength Prepakt concrete encasements. The extremely low shrinkage of Prepakt concrete provided a solid monolithic structure with a tight bond to reinforcement and old concrete.

Advantages in time saved and ease of placement have made Intrusion-Prepakt methods and materials particularly well suited to all types of bridge reconstruction and rehabilitation projects. For further information and descriptive engineering report, contact Intrusion-Prepakt, Inc., Room 568-J, Union Commerce Building, Cleveland 14, Ohio. In Canada: Intrusion-Prepakt, Ltd., 159 Bay Street, Toronto, Canada.

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Intrusion and Prepakt are trade marks of Intrusion-Prepakt, Inc., whose methods and materials are covered by U. S. Patents Nos. 2313110, 2655034, 2434302 and others, also patents pending.

ACI

News Letter Contents

September 1957

On the Cover—City of Seattle: Site of ACI Regional Meeting, November 4-6. Skyline view showing Elliott Bay; Alaskan Viaduct along waterfront; and the Cascade Mountain Range in the background.

—*Photograph courtesy Seattle Chamber of Commerce*

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ACI Regional Meeting in Seattle

Schedule of Events Set for Nov. 4-6

Complete program plans for the ACI 10th Regional Meeting scheduled for November 4, 5, and 6 at the Benjamin Franklin Hotel, Seattle, have been announced by the local committee.

The Technical Activities Committee will meet on Monday morning, November 4, followed by a joint luncheon with the Board of Direction at noon. The Board will convene on Monday afternoon.

Officially the conference will get under way on Tuesday, November 5, with registration starting at 9:30. An action-packed program for the 5th and 6th includes three technical sessions, a luncheon, banquet, and inspection trip. A special ladies program has been planned which includes luncheon at the Benjamin Franklin Hotel followed by a Grayline tour of Seattle on Tuesday afternoon, and a luncheon and fashion show at Frederick and Nelson's department store, Wednesday afternoon.

The technical portion of the program (listed on p. 4) will offer papers of particular interest to the Northwest area. The inspection trip scheduled for Wednesday afternoon includes the Boeing Developmental Center (the subject of one of the technical papers to be presented and an interesting prestressed concrete structure) as well as St. Edward's Church, a precast concrete job completed about a year ago, and which has received national recognition.

On Thursday, November 7, after the formal portion of the meeting is concluded, plans are being made to visit several concrete projects somewhat remote from Seattle such as some of the dams.

For those who have time to linger a little longer, Seattle offers much in the way of sightseeing;

points of interest are the Government Locks, second only to the Panama Canal in the Western Hemisphere; the Arboretum, part of a scenic lake drive; Lake Washington floating bridge; and the United States Puget Sound Naval Shipyard, where many of the Navy's largest carriers and warships are berthed, is just across the bay.

R. H. Lochow, Portland Cement Association, Seattle, is chairman of the local committee in charge of the regional meeting. Roscoe C. Hildebrandt, PCA, Seattle, is in charge of publicity. Other chairmen of subcommittees for the conference are: Fred H. Rhodes, Jr., University of Washington, registration and hospitality; Alfred L. Miller, University of Washington, educational contact; Ray A. Young, Darex AEA Division, Charles R. Watts Co., finance; Peter H. Hostmark, structural engineer, technical program; William P. Jackson, Glacier Sand & Gravel Co., inspection trips; Leland L. Sphar, Concrete Products Association of Washington, ladies entertainment; Abbott W. LaGrelius, banquet, and Donald G. Metcalf, hotel reservations and luncheons.

SEATTLE PROGRAM

Papers Scheduled for Presentation at Regional Meeting

High-Density Concrete for Shielding Atomic Energy Plants

Harold S. Davis, General Electric Co., Richland, Wash.

Synopsis—Massive concrete walls and slabs are used in atomic energy plants to protect personnel and instrumentation from the harmful effects of nuclear radiations. The thickness of a shielding wall can be decreased by incorporating heavy aggregates, such as magnetite or iron slags, in the concrete mix. Data are presented on the composition, properties, and cost of various types of high-density concrete. Special problems of interest to the engineer are discussed, as well as fields of development and research associated with the utilization of concrete in structures exposed to nuclear radiation and high temperature effects.

Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings

Jack R. Benjamin and Harry A. Williams, Stanford University, Palto Alto, Calif.

Synopsis—A number of scale model one-story reinforced concrete shear walls containing openings were tested at Stanford University as a part of a major study of shear walls sponsored by the Corps of Engineers. This paper reports on the observed behavior, theoretical studies, and recommendations for analysis of such walls. Studies of the influence of variations in reinforcing are included. One series of tests shows that proper reinforcing can make a wall with an opening equal to 22 percent of the wall area behave essentially the same as a solid wall.

Studies on Plastic Flow Characteristics of Lightweight-Aggregate Concrete

T. J. McClellan, Oregon State College, Corvallis, Ore.

Synopsis—The ultimate goal of this test program is a quantitative determination of the plastic flow occurring under compressive stress in a variety of expanded shale aggregate concrete mixes. There is need for this data to enable a reasonable prediction of loss of stress in tendons of pre-tensioned bonded lightweight-aggregate concrete.

At the present time there is insufficient evidence to make recommendations or to draw conclusions. Not enough tests have been completed nor have a wide enough variety of mixes been tested to warrant recommendations. There is, however, considerable evidence to in-

LAKE WASHINGTON'S FLOATING BRIDGE, opened to traffic in 1940, spans the lake from Seattle to Mercer Island. This four-lane concrete highway on huge concrete pontoons is the largest floating structure in the world (6560 ft long). Plans are being drawn for a second Lake Washington floating bridge, and preliminary surveys are currently underway for a floating bridge across Puget Sound. Cascade mountain range, which extends from California across Washington and Oregon, is in the background

dicate that there is a fairly definite point of strain equilibrium obtainable for the mixes tested, and forseeably for any other mix, and the plastic flow factors are not excessive for these mixes.

The current test series is being made with 2 ft x 2 ft x 18 in. specimens loaded in compression in specially fabricated steel yokes, with both yoke and specimen instrumented with SR-4 electric strain gage and 10-in. Whitemore mechanical strain gage. Verification between the two gage types is reasonable considering the difference in precision between the two types of gages.

Reinforced Masonry as Developed in the Northwest

Verne Frese, Layrite Concrete Products, Seattle, Wash.

Synopsis—A brief comment on the system of reinforced masonry construction developed in the Pacific Northwest, including the use of column-block columns and block lintel and bond beams. This system produces buildings with the structural integrity of reinforced concrete, combined with the economy, appearance, and utility of unit masonry. Some of the advantages as well as the problems are presented.

Warping of Reinforced Concrete Due to Shrinkage

Alfred L. Miller, University of Washington, Seattle, Wash.

Synopsis—Objectionable deflections and deformations of structural elements, especially relatively thin slabs and shallow beams of buildings, usually are ascribed to faulty workmanship, creep, or plastic flow which actually may be evidence of warping. Warping due to differentials of temperature or moisture is well known but because the similar effect of shrinkage is little understood, it has been relegated to the realm of uncertainty by attributing it to creep or plastic flow.

Experimental investigation of the subject reveals that the warping of reinforced concrete members due to shrinkage during the period of seasoning is an inherent characteristic of the

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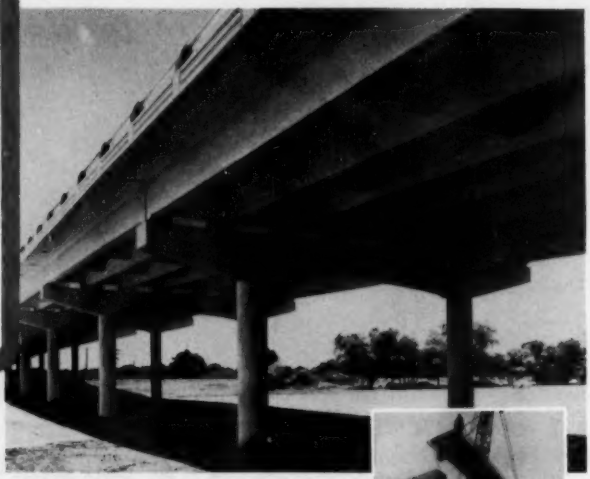

Wisner Drive Overpass, New Orleans, La. G. A. Heft & Company, Engineers. Keller Const. Corp., general contractors. Built for City of New Orleans. Photo courtesy Lone Star Cement Corporation.

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materials that can be anticipated and controlled. A practical theory is developed by which the amount of warping can be predicted and provision made for its reduction or elimination.

Prestressing of Cylindrical Concrete Shell Structures

John B. Skilling and John V. Christiansen, Worthington & Skilling, Seattle, Wash.

Synopsis—The reasons for prestressing cylindrical concrete shell structures and the conditions under which prestressing might be economical are discussed. Methods to use in prestressing are covered, both as to type (pre-tension or post-tension) and placement of the prestress material.

Two structures, designed by the authors, recently completed in the Northwest, employing both cylindrical shells and prestressing are given as examples. The general configuration of the structures and their stress distribution are illustrated. The advantages of prestressing these two projects are discussed.

Economic Factors in Prestressed Lift-Slab Construction

Edward K. Rice, T. Y. Lin and Associates, Los Angeles, Calif.

Synopsis—Why most lift slabs must be prestressed to obtain certain acceptable performance is discussed. Items covered include cost trends in conventional versus lift-slab construction, factors affecting the economy of prestressed lift-slab construction including building layout, structural framing schemes, use of precast columns, connections, prestressing layout. Results of load tests and long-term deflection measurements of prestressed lift-slab structures are given.

Manufacture and Application of Colored and Textured Precast Concrete Units

Otto Buehner, Otto Buehner and Co., Salt Lake City, Utah

Synopsis—The tremendous range of colors and textures made available to precast concrete through the Mo-Sai system is described and illustrated. The production of module sized units as well as intricate grill and inlay work and other diversified special applications are described. Technical aspects of the manufacture of Mo-Sai are considered.

Design and Construction of a Precast and Prestressed Concrete Building for the Boeing Airplane Co. Developmental Center

Alfred T. Waidelich, Austin Co., Cleveland, Ohio, and Arthur R. Anderson, Anderson, Birkeland and Anderson, Tacoma, Wash.

Synopsis—The main building of the Boeing developmental center is a two-story precast and prestressed concrete structure 50 ft high. The structural framing consists of 40 x 60-ft bays for the second story and 30 x 40-ft bays for the first story. Structural design problems are discussed with reference to the development of a composite and continuous structure to resist service and seismic loads. Details of fabrication and erection are also discussed.

The Deas Island Tunnel

Norman D. Lee, Foundation of Canada Engineering Corp., Ltd., Vancouver, B. C.

Synopsis—Illustrated discussion of this tunnel now under construction across the south arm of the Fraser River near Vancouver, B. C. It is being planned as a portion of a metropolitan freeway network and might be expected to be the link of this network which will ultimately form a freeway connection between Vancouver and Seattle. From portal to portal the tunnel will be about 2100 ft and it is being constructed by floating six 20,000-ton concrete elements and sinking them in place. Total cost of the project is in the order of $17,000,000.

Darex Diary

No. 14 of a series

by Henry L. Kennedy
Member, American Concrete Institute
Manager, Construction Products
Dewey & Almy Chemical Company

WATER-reducing agents improve *all* properties of portland cement concrete—strength, among others. Yet, this factor of increased strength has been so overemphasized that many concrete users are tempted to use water-reducing agents to reduce the cement content of the mix. This is a false economy. It ignores the possible harmful effects of reducing cement content and nullifies the other advantages offered by water-reducing agents and a low water-cement ratio.

Portland cement concrete is made of fine and coarse aggregate. Both are usually inert; both vary in quality. Portland cement paste, which also varies in quality, binds these into a rigid homogeneous mass. Aside from selective choice of raw materials, an effective way to control the quality of portland cement paste is to limit the water content. The less water, the higher the quality; the more water used, the lower the quality. Here, as with glue, maximum concentration gives maximum strength. Dilute with water and you lose strength. Hence the value of water-reducing agents.

Decades ago Duff Abrams termed this the "water-cement ratio law." While, as we have seen, most applications of the law emphasize compressive strength, Abrams stressed that the mix must be workable and placeable before the law can be properly applied. The American Concrete Institute's Committee 613 report also recognized the contribution low water-cement ratios make to the durability of concrete. Their findings were presented in the form of tabulated exposure conditions vs. recommended maximum water-cement ratios.

Let's look at some of the other benefits gained by a low W/C in concrete—plastic as well as hardened. Bleeding, segregation and honeycombing are reduced, to give better handling and placing characteristics in plastic concrete. Water gain under the aggregate, laitance on the surface of a pour, sand streaking, and plastic shrinkage cracks are virtually eliminated. Green shrinkage is substantially cut.

With reduced bleeding, segregation and honeycombing, concrete hardens with far fewer entrapped air voids. Verbeck of the Portland Cement Association showed that "gel pores," too, are reduced in number, along with the capillary pores to which absorption characteristics in hardened concrete are primarily attributed. Concrete homogeneity is increased, and permeability —especially to water under pressure—is greatly reduced.

This reduced permeability resulting from lowered water content is probably the greatest single influence on the quality of concrete. It brings about an improvement in durability, particularly frost resistance, over and above that accomplished by the entrainment of fine, well-dispersed air bubbles. Reduced permeability also means greater resistance to salt, alkalis and sulfates; and improved resistance to disintegration due to constant immersion in sea water. And because less water can penetrate the concrete to dissolve the soluble salts, leaching and efflorescence are minimized.

Low water content also cuts drying shrinkage and green shrinkage, thus eliminating many construction headaches due to changes in volume. Low W/C reduces early expansion in concrete cured under water, which in the hardened concrete means increased resistance to alternate wet and dry, hot and cold exposures. Sufficient data exist to support this.

In any application of water-reducing agents and the water-cement ratio law, we must consider this full range of benefits. An increase in strength must not be overemphasized merely to rationalize a lowering of cement content, for this can adversely affect other properties of concrete. Never, under any circumstance, should cement content be less than 4 sacks/cu. yd. for normal 1½" top-size aggregate.

* * * * * * * * * * * * * *

Backed by over 25 years' experience, Dewey and Almy's water-reducing agent, in liquid form—"WRDA"—offers a triple benefit: water content can be reduced as much as 20%; compressive strength is increased up to 30%; over-all quality of the concrete is greatly improved.

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Forty-six units joined for ACI's

precast folded plate roof

ON-THE-JOB PRECASTING of unique folded plate roof sections commanded the camera's attention at the construction site of ACI's new headquarters building in Detroit. Several of the completed roof slab sections are shown above, stored on the ground awaiting installation. Roof erection began in mid-August, with completion of the precasting, completed concreting of supporting first-floor corridor walls, and final stages of roof erection anticipated about September 10.

This imaginative use of concrete by architect Minoru Yamasaki implements his view that "concrete is inherently more natural to slab type construction than to post and beam . . . " The 4-in. thick folded slab roof is cantilevered from concrete bearing walls of the interior corridor, which are further stabilized by the "box" of the monolithic concrete basement.

The roof was cast on the ground in 46 segments, each a double reverse fold, with a closed, triangular-shaped beam box beneath the interior end of the slab segment. These triangular boxes will be joined at their apexes along the center line of the roof by concreting and welding of reinforcement to form X-shaped beams over the building's central hall. The roof will be pierced by skylights over the corridor between the beams. Slabs have built-in scupper, electrical conduits, and two loops for lifting.

Assembly of the reinforcing steel was done on a wooden jig (picture

Airborne roof slab segment has just been lifted from the form and will be stored on the ground at building site until erection

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Assembly of the reinforcing steel was done on a wooden jig (picture

Airborne roof slab segment has just been lifted from the form and will be stored on the ground at building site until erection

Removal of finished slab from its casting bed by crane. Lines are attached to two lifting loops that were cast in the slab

below) the same size and shape as the form used for casting. More than 70 detailed types of reinforcing and metal inserts were used, with three #9 bars forming the main longitudinal reinforcement of the 23½-ft span. Steel plates ⅜ in. thick running along each edge of the roof sections will be welded to join the sections and provide transverse moment connection. Two rows of dowels spaced 4 in. on centers were welded to these metal plates and embedded in the concrete slab. Reinforcing steel for the 4½-ton panels was supplied by Concrete Steel Corp., Detroit.

Concreting the folded plate sections

Three forms were built on a wooden base resting on footings, and two roof sections were cast daily, weather permitting. An elevated working platform was built beside each form so that ready-mix trucks could pull up and discharge concrete on it. Workmen standing on the raised platform then shoveled concrete into the form.

Ready-mixed concrete made with Peerless Type I cement was supplied by Frank J. Knight Co., Detroit, at a slump of 3 to 4½ in. Aggregate produced by American Aggregates Corp., Detroit, was ⅜-in. maximum size, premium grade gravel, with total deleterious material reduced to 4 percent by heavy-media separation. Nominal mix proportions were 1:2.07:2.48, with a W/C ratio of 5 gal. per bag, and a cement factor of 6½ bags per cu yd.

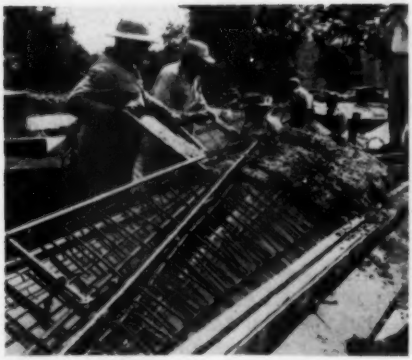
Pozzolith No. 8 was added to the mix to retard set in anticipated 90 F air temperature, and to accelerate strength development. Average 24-hr strength of the field-cured concrete was 2300 psi. Laboratory-cured specimens tested 4400 psi at 7 days, and 5300 psi at 28 days.

Finishing and curing

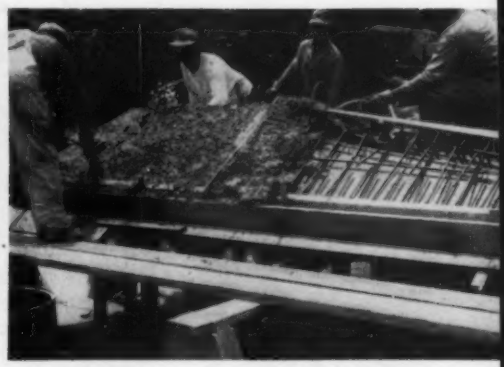
Slab sections were cast in inverted position, and the concrete was consolidated by vibra-

Reinforcing was assembled on a jig of exact size and shape of the folded slab form; three #9 bars are main reinforcement

Detail of reinforcing and form for beam box which was cast at the interior end of each slab section. Two boxes form X-beam



Placing concrete in form was by shovel from elevated platform where it had been unloaded by ready-mix truck. Vibration followed, ahead of finishing shown at right



Screeding and troweling provided surface finish. Portion to right was troweled since it will be exposed in roof overhang; remainder was screeded only, will be covered by interior plaster on the ceiling

tion, with conventional finishing as shown in the photographs. They were cured under wet burlap and Secoton, a white liquid membrane-forming compound manufactured by Surface Engineering Co., Wichita, Kan., was sprayed on the portion of the slab which will later form the interior ceiling of the building and will be covered by plaster. The underside of the overhanging portion of the roof was given a troweled finish (this portion shows uppermost in the photographs since slabs were cast in inverted position). Intermediate finishing was done on the triangular beam sections, with forms stripped there 2 hr after casting. After about 24 hr curing the slab sections were lifted off the forms (picture on page to left) and moved to storage area at the construction site. Water ponded in the folds of the slab and the beam box at the slab end aided in continued curing. Before erection the slabs were turned over and cleaned down according to standard procedures recommended by the Portland Cement Association.

Continued on p. 24

Finishing touches . . . final finishing of roof slab segment in the foreground, with reinforcement in place for casting the next slab on the adjacent form. At the right, rear, is a third section awaiting lifting from the form. Finished sections of the roof are in the background awaiting erection

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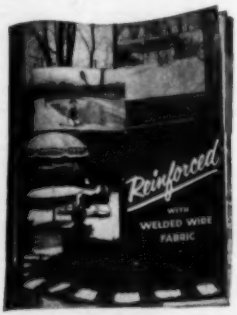

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SAY YES . . . WITH

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THE COLORADO FUEL AND IRON CORPORATION
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Positions and Projects

1957 ACI Book of Standards

The 1957 edition of the *ACI Book of Standards* (300 pp.) is now off the press. The book represents the most recent compilation of current ACI standards, recommended practices, and specifications.

Eleven ACI standards are compiled under one cover, incorporating such subjects as the evaluation of compression test results; building code requirements for reinforced concrete; design and construction of concrete chimneys; winter concreting; selection of proper proportions for concrete; and the measuring, mixing, and placing of concrete. The book also discusses the application of portland cement paint, concrete pavements and bases, precast concrete floor units, construction of concrete farm silos, and the application of mortar by pneumatic pressure.

The book is available through ACI at $4 per copy for nonmembers and $2 to ACI members.

Cousins joins ACI staff

Charles L. Cousins, a 1957 graduate of Michigan State University, recently joined the ACI staff as advertising manager of the ACI JOURNAL with additional public relations duties.

Mr. Cousins majored in journalism at MSU. Prior to joining ACI he was advertising manager and assistant editor of the *Antrim County* (Michigan) *News* for a short time.

Steinman to receive Levy medal

Dr. David B. Steinman will be awarded the Louis E. Levy Medal in recognition of his outstanding paper "The Design of the Mackinac Bridge for Aerodynamic Stability" which appeared in the December 1956 issue of the *Journal of The Franklin Institute*. Formal presentation of the award will be made at the Institute's annual Medal Day ceremonies on October 16.

This will be the fourth award Dr. Steinman has received for his design of the Mackinac bridge. Others are the J. Lloyd Kimbrough Gold Medal from the American Institute of Steel Construction; the George W. Goethals Medal from the Society of American Military Engineers, and the Gzowski Gold Medal from the Engineering Institute of Canada.

4th edition of ACI Manual of Concrete Inspection released

The latest edition of the pocket-size *ACI Manual of Concrete Inspection* is now available. General editorial revisions have been made throughout the text and the sections dealing with mix proportioning, winter concreting, and hot weather concreting have been brought up-to-date.

Written and bound for use at the construction site as well as the laboratory and office, the text is brief and readable, and interprets the policies as set forth by authorized bodies. The manual covers concrete fundamentals of the latest developments in construction as well as problems and techniques in concrete inspection.

The manual was prepared by ACI Committee 611, Inspection of Concrete, under the leadership of Lewis H. Tuthill. Copies are available through ACI headquarters at $3.50 each to nonmembers—$1.75 to ACI members.

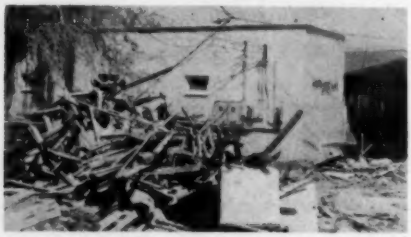
New BRAB committee appointed

William H. Scheick, BRAB executive director, announced the appointment of a Building Research Advisory Board committee formed to advise the Federal Housing Administration on its technical studies program.

Albert G. H. Dietz, professor of building engineering and construction, Massachusetts Institute of Technology, is chairman of the group. Two ACI members, Charles M. Goodman, Charles M. Goodman Associates, Washington, D. C., and J. Neils Thompson, director, Balcones Research Center, and professor of civil engineering, University of Texas, Austin, are members of the committee.

Professor Thompson is chairman of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight-Aggregate Concrete, and a member of ACI Committee 209, Volume Changes and Plastic Flow in Concrete.

CONCRETE *stands up to hurricane Audrey*



AERIAL VIEW OF REINFORCED CONCRETE courthouse (left) at Cameron, La. ,4 days after hurricane Audrey had devastated the area. The building came through unscathed in contrast to the surrounding destruction. Concrete masonry building (right) was one of the few structures that escaped destruction. Piles of scattered lumber are mute evidence of the fate of the majority of the buildings in the southern community

Out of the Gulf of Mexico on the morning of June 27, 1957, roared the first hurricane of the season, striking with winds up to 110 mph and causing a 10-ft tidal wave to sweep inland. News reports suddenly focused the attention of the nation on the little town of Cameron, La. As rescue workers brought out survivors, a commanding story on the strength of concrete emerged—how the parish courthouse sheltered storm victims from wind and water. Originally built in 1937, the building was designed of reinforced concrete especially to provide the town with a place of refuge during hurricanes.

Following the storm, the concrete courthouse stood undamaged in the midst of scattered timbers, twisted buildings, and washed-up fishing vessels. Although water had filled the basement and risen to the top of the 7-ft high front steps, the base of the building was sound. As the only habitable building in the area, it became the center of rescue and clean-up operations. Wood buildings in the town, if not demolished by the wind, had in many cases been swept off their foundations or overturned by the tidal wave. Several concrete masonry structures, including the ice house and a fairly new structure containing the Marine National Bank and a supermarket, were relatively undamaged.

The remarkable way in which the courthouse withstood both wind and water is striking proof of the importance of storm-resistant design in areas exposed to hurricane or tornadoes. Wide footings for the courthouse were extended down through sand and shell to hard clay. The reinforced concrete walls were 12 in. thick, buttressed by battered pylons and braced by concrete floors and roof. The courthouse was designed by architect Herman J. Duncan; A. Farnell Blair, of Lake Charles, La., was contractor.

Gerstell elected president

R. S. Gerstell has been elected president of Alpha Portland Cement Co., Easton, Pa., to succeed J. F. Maggee, executive officer, who has been named chairman of the board. Officers re-elected to 1-year terms are: J. D. McKelvy, vice-president; N. O. Wagner, vice-president in charge of sales; K. T. Wright, secretary and treasurer; and H. F. Stepanek, assistant secretary and assistant treasurer.

Radiation shielding properties of concrete to be studied

The U. S. Navy Bureau of Yards and Docks has awarded a contract to Walter Kidde Nuclear Laboratories, Inc. to perform radiation shielding studies. According to W. E. Kelley, president of the firm, the purpose of the work is to develop basic and engineering data on shielding properties of readily available materials. Studies will be made on different thicknesses and types of soils and concrete to determine their shielding properties against neutron radiation.

How to Choose Waterstop for optimum performance in concrete joints

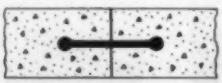
Basic Design And Resilience Most Important Factors

The principal function of waterstop is to keep concrete joints watertight where hydrostatic water pressure is present. To be effective, and to perform its function under widely varying conditions, the waterstop must:

1. **Be designed in such a way that it will maintain a "pressure seal" when the joint is opened up or compressed, or when hydrostatic pressure is exerted against it.**

2. **Be made of a material that is inherently stable and resilient . . . that will retain its resiliency and strength under wide ranges of temperature.**

There is general agreement by many governmental and private specifying authorities, after years of testing and actual installation, that the dumbbell design of waterstop (below) is mechanically superior to any which has been developed to date. The design provides a self-sealing action, because as the concrete contracts and the joint opens up, the outer edges of the dumbbell bulbs become more tightly engaged with the concrete, insuring a tighter seal as the tension increases, due to movement either in the joint or increasing water pressure on one side of the joint. In effect, the greater the longitudinal pull or pressure on one side, the more tightly the dumbbell ends are pulled and squeezed against the concrete. The simpler dumbbell design of the rubber waterstop allows full strength and contact with the concrete surrounding the waterstop. The larger design also provides for maximum strength to resist higher pressures on the web of the waterstop across the joint opening.



9" DUMBBELL TYPE EMBEDDED IN CONCRETE WITH A CONSTRUCTION JOINT

9" HOLLOW BULB TYPE EMBEDDED IN CONCRETE WITH AN EXPANSION JOINT

Rubber and vinyl are the most commonly used waterstop materials. For the majority of applications, rubber is the most satisfactory material. Being a thermosetting material, rubber is more resilient and "live" . . . will maintain a constant pull against the retaining edges (bulbs) as the joint opens up or water pressure increases. Vinyl is a thermoplastic compound and tends to take a "set" after it has been stretched, will float in the joint cavity, and have less resistance to the passage of water. When higher temperatures are present, such as in oil storage tanks, where oil is kept at temperature around 150° F., the vinyl material, unless specifications are rigidly written, will soften and lose strength, causing a failure of the waterstop.

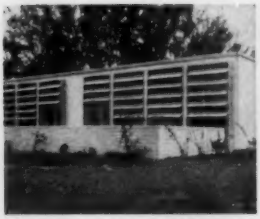
FIELD SPLICING OF DUMBBELL TYPE

Servicised Products Corporation has developed a new Union which provides a simple method of joining the ends of dumbbell waterstops, making it just as fast and easy to field splice rubber and neoprene waterstop as the joining of the polyvinylchloride types.

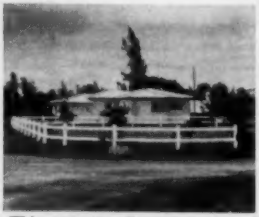
The Union is made in the same cross-section, and from the same material as the waterstop. It is hollow, except for a solid web at the center. After adhesive is applied to the waterstop ends, they are inserted in the Union and pushed against the centering web. The splice is then clamped together until the adhesive has set. This completes the splice.

Dumbbell type rubber and neoprene waterstop are fully described in a special Waterstop Circular available from Servicised Products Corporation upon request. The Union and an interesting new development, Split Type, are also illustrated and described in the circular. Write for it today.

SERVICISED PRODUCTS CORPORATION
6051 W. 65th Street • Chicago 38, Illinois



Large precast window frames create a shadow-box effect for this school.



This attractive fence is made of precast posts and rails.

Combined window, door and panel frames for a Miami Beach hotel.

Placing prestressed deck slab on new bridge leading to the site of Lehigh's new Miami cement plant.

Miami's first prestressed concrete bridge which crosses the Tamiami Canal. Built in 1953.

Placing concrete in double tee forms at Lewis casting yard.

"Fast, Economical Production"

with

LEHIGH EARLY STRENGTH CEMENT

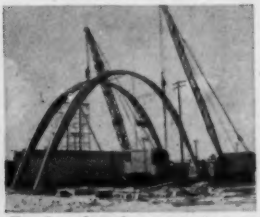
Since 1948, the Lewis Manufacturing Company of Miami, Florida, has been making a wide variety of concrete products. Pictured here are just a few of them—along with some of the jobs in which they have been used.

"Our entire operation," writes Mr. Lewis, "has always been geared to the use of Lehigh Early Strength Cement and live steam curing. This fast, economical production method enables us to give our customers better service, at lower cost."

LEHIGH PORTLAND CEMENT COMPANY

Allentown, Pa.

- LEHIGH EARLY STRENGTH CEMENT
- LEHIGH PORTLAND CEMENT
- LEHIGH AIR-ENTRAINING CEMENT
- LEHIGH MORTAR CEMENT



Erecting 2-section precast parabolic arches for a Miami high school auditorium.

Placing precast wall panels for television station WCKT.

Precast fins, 11′ x 10″ x 2″, make an interesting patio enclosure.

1902 CU YD OF CONCRETE IN 8 HR was the "record" set during construction of the prestressing plant of American-Marietta's Lewistown Pipe Division near LaGrange, Ill. The plant has a $2,900,000 contract with the Illinois State Toll Highway Commission to furnish more than 52,000 tons of prestressed concrete bridge girders for the Tri-State Route.

The finished plant includes one prestressing bed 691 ft long, and a second bed 661 ft long, containing a total of 200 tons of reinforcing steel and 4000 cu yd of concrete. Except for surface capping, each bed was cast continuously to form the monolithic self-contained structures required because of poor foundation soil conditions at the site. The beds are designed for a maximum moment of 45,000,000 in.-lb. M. A. Lombard & Son Co. was contractor for the work, and Dolese & Shepard Co. supplied the ready-mixed concrete.

PCA opens Detroit office

Opening of a Portland Cement Association branch office in Detroit has been announced by J. Gardner Martin, Michigan district engineer for the association. The Detroit office will supplement the association's main Michigan district headquarters in Lansing.

Several personnel changes and additions also have been announced. T. M. Reynolds, who has served as paving and public works engineer in the Detroit area since 1949, has been appointed statewide paving engineer with headquarters in Lansing. J. E. Kratzer, general field engineer for PCA since 1949 in eastern lower Michigan, has been named paving and public works engineer in Wayne County. R. A. Backus, who has served since 1953 as housing and products engineer, becomes paving and public works engineer for the suburban Detroit area.

Recent additions to the PCA staff in Michigan include William C. Krell, formerly chief structural engineer with Smith, Tarapata & MacMahon, who will serve as Detroit area structural field engineer, and Willard O. Ashmore, formerly employed on the Grand Rapids city engineering staff, who has been appointed general field engineer for northern lower Michigan.

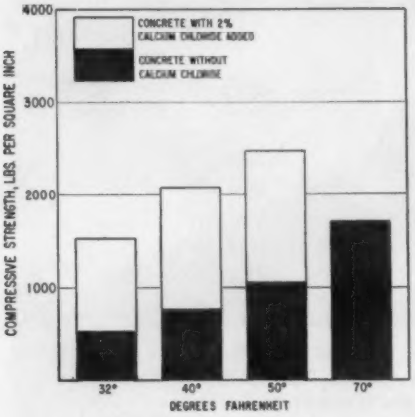
ASTM elects officers

At the close of the recent 60th annual meeting of the American Society for Testing Materials, Rudolph A. Schatzel turned over his ASTM presidential duties to Richard T. Kropf, newly elected president of the organization. Mr. Kropf is associated with Belding Heminway Co., New York. Frank L. LaQue, International Nickel Co., will serve as vice-president.

PCA promotes Sillars

The appointment of Robertson Sillars as manager of the Publications Bureau of the Portland Cement Association, Chicago, was recently announced by James D. Piper, vice-president for promotion. He succeeds J. L. Schneider who will devote full time to his duties as assistant secretary of the association.

3 DAY STRENGTH OF CONCRETE AT VARIOUS TEMPERATURES



GRAPHIC PROOF

Why you need SOLVAY CALCIUM CHLORIDE *when temperatures drop . . .*

Compare 3-day-old concrete with and without SOLVAY Calcium Chloride. At 32°F., the concrete containing calcium chloride is 222% stronger—at 40°F., 182% stronger—at 50°F., 141% stronger!* Similar gains are produced at 1, 2 and 7 days. By the addition of 2% low-cost SOLVAY Calcium Chloride to your mix you can maintain normal working schedules. Protection time is cut . . . every operation—finishing to form removal—moves on time.

In any season, SOLVAY Calcium Chloride gives you **better** concrete. It increases **both** early and ultimate strength. In addition, it permits reduction of water-cement ratio which results in denser, more moisture- and wear-resistant concrete.

*From Highway Research Board Proceedings.

SOLVAY CALCIUM CHLORIDE speeds but does not change the normal chemical action of portland cement. Its use in cold-weather concrete is recommended or approved by leading authorities, including A.C.I. and P.C.A.

SOLVAY

New handy 25 lb. bag
Easy to use
Assures accurate measure

Write for calcium chloride fact books at no cost!

SOLVAY PROCESS DIVISION

Allied Chemical

ALLIED CHEMICAL & DYE CORPORATION
61 Broadway, New York 6, N. Y.

Topping elected president of BRI

Charles H. Topping, senior architectural and civil consultant, E. I. du Pont de Nemours and Co., Wilmington, Del., has been elected president of the Building Research Institute. Harold L. Humes, Baldwin-Hill Co., Trenton, N. J., was named vice-president.

Fellowship established in memory of A. E. Cummings

This month a talented student will begin his graduate studies in soil mechanics at the University of Illinois as the first recipient of the newly created "Raymond Concrete Pile Co. Fellowship in Memorial to A. E. Cummings."

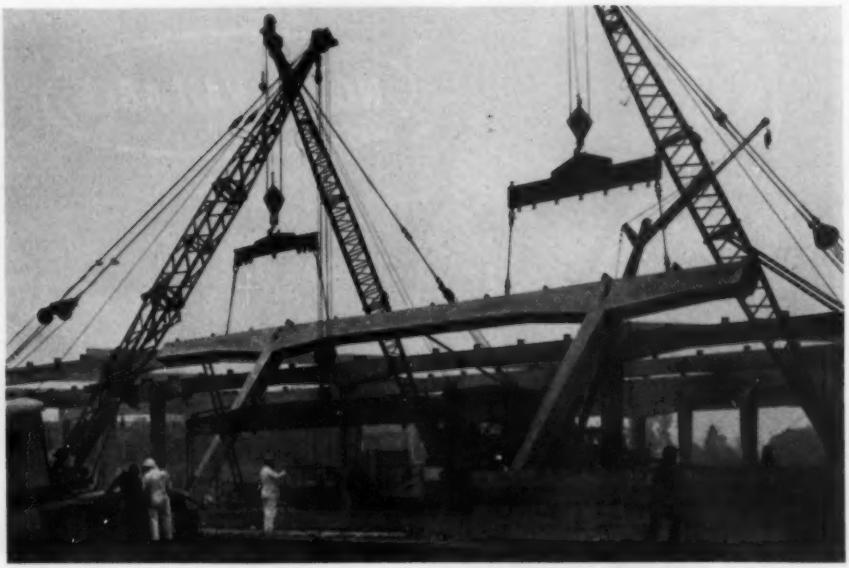
The scholarship has been established by Raymond Concrete Pile Co. in memory of the late director of research and company director, A. E. Cummings.

Mr. Cummings, who died in 1955, was a pioneer in the field of soil mechanics. He took an active role in its development and practical application to foundation work since its introduction to the engineering profession over 35 years ago.

A member of ACI since 1924, he was active in committee work. He was a member of the Board of Direction in 1950-1952; chairman of the joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete; member of ACI Committees 318, Standard Building Code, and 324, Precast Reinforced Concrete Structures. He was ACI representative on ASA Sectional Committee A-56, Building Code Requirements for Excavations and Foundations.

U. S. Government needs engineers

The United States Civil Service Commission recently issued an announcement for engineer examination with no closing date. The examination will be used to fill positions in various federal agencies in the United States and foreign countries. At the present time, the federal government employs some 50,000 professional engineers in a wide variety of engineering fields throughout the country. Further details are available from the U. S. Civil Service Commission, Washington 25, D. C.



PRECAST BENTS being hoisted in sections 100 ft long. Columns are on 50 ft centers with 25 ft cantilever each side of the column. Connected with a bolt, they form a continuous frame 300 ft long. The project is the instrument building for Varian Associates, Palo Alto, Calif.; Michael Gallis, architect, and Isadore Thompson, San Francisco, consulting structural engineer

Arkansas Cement Corp. begins construction of cement plant

A multimillion dollar cement plant will be built at Foreman, Ark., by the Arkansas Cement Corp., which was organized early in July. Construction will start next month and October 1, 1958, has been set as the target date for initial cement production. Capacity of the plant as presently planned will be 1,400,000 barrels of cement annually.

Engineering and construction work on the new plant is being undertaken by Kaiser Engineers, a division of Kaiser Industries, Oakland, Calif.

Keystone personnel changes

Keystone Portland Cement Co. has announced that Edwin K. Borchard, associated with their organization for more than 22 years, will serve as technical consultant for the company in the future. John F. Hall is the newly appointed manager of the technical department with headquarters in New York City.

Cavanagh made deputy director

Kevin Cavanagh has been appointed deputy director of the Cement and Concrete Association of Australia. Mr. Cavanagh was formerly materials engineer of the association.

Marks joins Houston consulting firm

C. R. Marks, who has directed engineering studies or designs on approximately 20 large dams, has joined the staff of Lockwood, Andrews & Newman, consulting engineers in Houston. Formerly he was vice-president, in charge of the Houston office, of Ambursen Engineering Corp. His projects include the Whitney Dam and powerhouse for the Corps of Engineers, the San Jacinto River Dam at Houston, and Weseley E. Seale Dam near Corpus Christi. Before going to Texas, Mr. Marks designed bridges for the Santa Fe Railroad, spillways for the U. S. Bureau of Reclamation, and various structures for the Tennessee Valley Authority.

Test after test, made by various independent research organizations, conclusively prove that DURAJOINT Waterstops are the only waterstops available on which you can stake the safety of your design and reputation. The results of these tests plus the many other advantages offered by DURAJOINT Waterstops are concisely stated in our new Waterstop Manual No. 457. Check and mail the coupon below for your free copy . . . it will make an excellent addition to your waterstop file.

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Carbonell represents ACI at Seventh Pan-American Highway Congress

ACI was represented by Celso A. Carbonell at the Seventh Pan-American Highway Congress which was held in the City of Panama Aug. 1-10, 1957.

Mr. Carbonell has been an active member of ACI since 1951. He now serves as a reviewer of Spanish bulletins on cement production for the "Current Reviews" section of the JOURNAL. As president of the Panamanian Society of Engineers and Architects, he attended the ninth regional meeting of the American Concrete Institute in Montreal last October. He is presently employed as regional manager in Panama City for Sika Chemical Corp.

ASTM committee actions

Committee activities were a prominent part of the American Society for Testing Materials' annual meeting at Atlantic City in June. ASTM Committee D-4, Road and Paving Materials, reported extensive activity in the development and refinement of many test methods. The committee recommended withdrawal of four specifications covering granite block and filler materials which are no longer used.

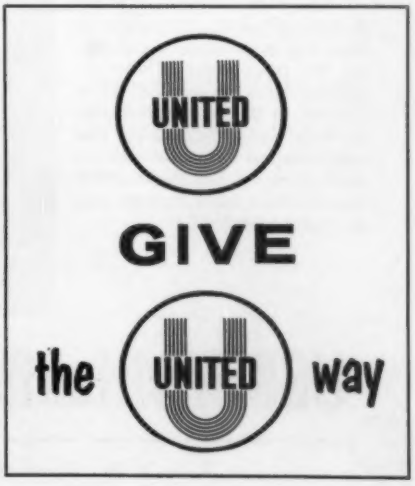
The report of Committee C-1 on Cement revealed progress in refining existing procedures and in filling in gaps where additional test methods are needed. Noticeable gains have been made toward international standardization of specifications. In regard to the operating procedures of the Cement Reference Laboratory, it has been recommended that the inspection be extended to cover concrete as well as cement testing and that the inspection should be provided at more frequent intervals than in the present program.

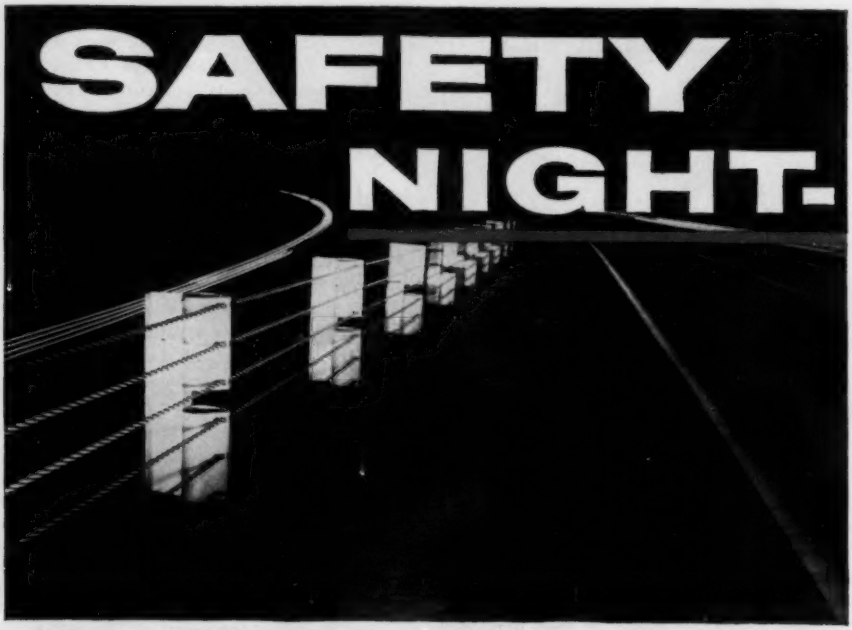
Committee C-9 on Concrete and Concrete Aggregates reported the preparation of specifications and test methods for pozzolanic materials used to reduce expansion of concrete caused by alkali-aggregate reaction. Extensive study of the subject of abrasion resistance of concrete by the committee has resulted in development of a proposed method known as the shotblast method. Revisions of the present specification for curing paper (C 171) have been agreed upon in subcommittee. In this same field, work will be initiated on specifications and methods of test for cotton mats for curing concrete. The committe will continue the study of pore structure of aggregates.

World Conference on Prestressed Concrete

The World Conference on Prestressed Concrete held at San Francisco was attended by nearly 1200. Registration included representatives from 30 countries. The program featured reports on practice in prestressed concrete design and construction in all major countries. The growing use of this technique in this country was reflected by papers reporting on prestressed concrete construction in all parts of the United States, and in particular, the adaptation of pre-tensioned prestressed concrete factory-made products for many applications heretofore only possible or economical in timber or structural steel.

ACI cooperated along with other organizations in the sponsorship of the one-week conference. T. Y. Lin, professor of civil engineering at the University of California, Berkeley, was chairman of the general arrangements committee. Thor Germundsson, Portland Cement Association, Chicago, and chairman of ACI-ASCE Committee 323 on prestressed concrete, represented ACI on the program and advisory committees.



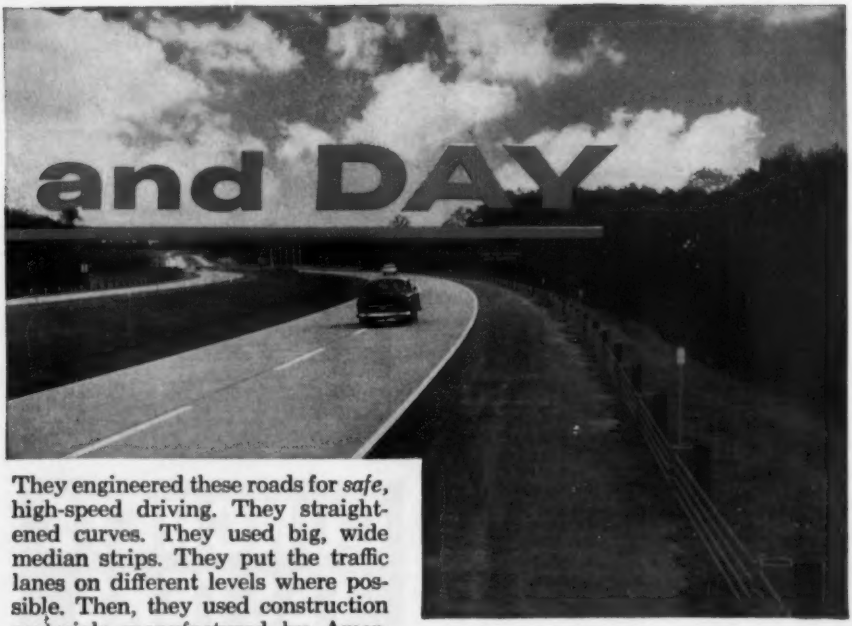


ALONG THE ROAD. Day and night, Multisafty Cable Guard stands guardian on America's newest highways, giving two-fold protection from serious accidents. It restrains vehicles from plummeting off the berm. In addition, its resilient combination of strong, steel cables and spring-steel offset brackets cushions the shock of any collision . . . helps prevent damage to the vehicle and injury to passengers.

ECONOMY IN CULVERTS.

American Welded Wire Fabric brings the same service-lengthening benefits to culverts as to pavements. Concrete, reinforced with Welded Wire Fabric, is the economical way to build culverts because this combination of materials resists abrasion, has the strength to withstand high stresses, and is easily installed.

USS Multisafty Highway Guard



They engineered these roads for *safe*, high-speed driving. They straightened curves. They used big, wide median strips. They put the traffic lanes on different levels where possible. Then, they used construction materials manufactured by American Steel & Wire to enhance the basic safety designed into the road.

Materials such as Multisafty Highway Cable Guard which protects the sides of the road . . . American Welded Wire Fabric which lengthens the life of the pavement, keeps it smooth, easy-riding, and safe . . . and specially designed American Road Joints which smooth the transition from slab to slab.

These are essential materials for any truly modern highway. Use them in your roads . . . in your city streets. Send the coupon for complete information.

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PACIFIC COAST DISTRIBUTORS
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UNITED STATES STEEL EXPORT COMPANY, NEW YORK

IN THE PAVEMENT A smooth pavement is a *safer* pavement. And American Welded Wire Fabric and American Road Joints help assure extra years of smooth, safe riding on great new highways such as the Ohio Turnpike, the Pennsylvania Turnpike, the Indiana Toll Road, and others.

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- ☐ American Beam-type Highway Guard
- ☐ American Wire and Strand for Prestressed Concrete
- ☐ Road Manual

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Engineers and Consultants
DESIGN AND SUPERVISION OF CONSTRUCTION
REPORTS—EXAMINATIONS—APPRAISALS
MACHINE DESIGN—TECHNICAL PUBLICATIONS
BOSTON NEW YORK

Building committee report

Continued from p. 11

A late report from the ACI Building Committee shows the following contributions to the ACI Building Fund by categories as of July 31, 1957:

Cement producers	$ 69,512.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,176.22
Contractors	18,445.00
Ready-mixed concrete and aggregate industry	10,983.87
Admixtures manufacturers	7,258.93
Membership at large	5,051.74
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,520.00
Eastern Canada	1,496.90
Special contributions	1,209.08
Total	$194,238.74
Pledges in hand	7,300.00
GRAND TOTAL	$201,538.74

Henry L. Kennedy, Dewey and Almy Chemical Co., Cambridge, Mass., heads the Building Committee. Aiding him in year-long activity have been: A. Allan Bates, vice-president for research and development, Portland Cement Association; Robert F. Blanks, vice-president and general manager, Great Western Aggregates, Inc.; Roger H. Corbetta, president, Corbetta Construction Co., Inc.; and Raymond E. Davis, consulting engineer and professor emeritus, University of California.

Others on the committee are Harry C. Delzell, managing director, Concrete Reinforcing Steel Institute; Jay E. Jellick, manager, Portland Cement Information Bureau; Frank Kerekes, dean of faculty, Michigan College of Mining and Technology; William A. Maples, secretary-treasurer, American Concrete Institute; Douglas McHenry, director of development, Portland Cement Association; Walter H. Price, chief, engineering laboratories, U. S. Bureau of Reclamation; Charles H. Scholer, professor of applied mechanics, Kansas State College; and Charles S. Whitney, partner, Ammann and Whitney.

Kolm leaves Lehigh

Roger E. Kolm has taken a position with Modjeski and Masters, consulting engineers, Philadelphia. Mr. Kolm formerly was a professor in the civil engineering department at Lehigh University, Bethlehem, Pa.

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Meeting planned for new ACI Chapter

The Southern California Chapter of the American Concrete Institute plans its first general meeting early in November. Samuel Hobbs, secretary-treasurer of the chapter, reports good results from the first letter to ACI members in southern California inviting them to become charter members of the chapter. A similar invitation to non-ACI members in the area will be mailed shortly.

Powers honored by ASTM

T. C. Powers, manager, Basic Research Section, Portland Cement Association, Chicago, received the Sanford E. Thompson award at the annual ASTM awards luncheon in Atlantic City in June for his outstanding work in the field of concrete and concrete aggregates.

ASTM established this award in 1938 to be given to the author or authors of papers of outstanding merit on concrete and concrete aggregates which are presented at meetings of the society. Its purposes are to stimulate research, to encourage the extension of knowledge of concrete and concrete aggregates, and to recognize meritorious efforts.

Mr. Powers has had numerous papers published in the ACI JOURNAL and has received the Wason Research Medal from ACI on several occasions. At the present time he is a member of ACI Committee 212, Admixtures.

Nike concrete placed

First concrete for Nike foundations is being placed at Site Summit, high in the Chugach Mountains above the Artic Ski Bowl, guarding Fort Richardson, Elmendorf Air Force Base, and the Anchorage area in Alaska.

Simultaneously, other projects are underway in the Anchorage locale and at other guided missile sites in the Fairbanks region in interior Alaska.

Present Nike construction, under supervision of the U. S. Army Engineer District, Alaska, totals more than $21,000,000. Construction on the Nike installations in Alaska will bring to the northernmost rim of defense the guided missiles already deployed through the United States as a defense for industrial, highly populated, and strategic areas.

MARACONS are water-reducing admixtures for concrete. They promote more complete hydration of cement particles and permit a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. This means —

A. Lower Concrete Costs: —

1. Attain higher strengths without increasing cement content of a mix.
2. Maintain slump and workability at low W/C ratios.
3. Permit economical redesign of conventional concrete mixes.

B. Better Quality Concrete: —

1. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.
2. Achieve greater bond strength and higher durability factor.

The MARACONS also reduce water requirements in concrete mixes containing pozzolanic materials.

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Hot job schedules and falling temperatures used to turn contractors' hair snow-white overnight. Barbers and wives report this isn't nearly as great a problem today. News of Columbia Calcium Chloride has gotten around.

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You meet strength specs days ahead, pull forms for other jobs, get finishers on and off the site without profit-killing overtime. *Right now* is the perfect time to analyze your batching procedures, or to talk to your ready mix suppliers.

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Who's Who This Month

Frank Kerekes

"Review of Changes in the ACI Building Code Requirements for Reinforced Concrete," appearing on p. 185 of this month's JOURNAL, was prepared by ACI past president, Frank Kerekes, Dean of the Faculty, Michigan College of Mining and Technology, Houghton, Mich. Dean Kerekes was chairman of the committee which developed "Building Code Requirements for Reinforced Concrete (ACI 318-56)," now used as a standard in more than 2000 cities.

Dean Kerekes has devoted his life to the profession of education and engineering. He served 34 years on the civil engineering faculty at Iowa State College, Ames, and advanced to assistant dean of engineering before taking over his present role at Michigan College of Mining and Technology. While at Iowa State he also served as chairman of both the Ames City Plan Commission and the Iowa Building Code Council. In 1948 while chairman of the Iowa Building Code Council, Dean Kerekes was designated a "Second Century Leader" by the *Des Moines Register and Tribune*. He has been particularly active in ACI committee work and has been a frequent contributor of papers for JOURNAL publication. He is a member of ACI Committee 315, Detailing Reinforced Concrete Structures, and Committee 318, Standard Building Code.

ACI Committee 318

In response to requests for elaboration of certain sections of the Appendix on ultimate strength design in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI Committee 318 offers the first of a series of such explanations on p. 197.

Raymond C. Reese, consulting engineer, Toledo, Ohio, is chairman of the committee; J. P. Thompson, Portland Cement Association, Chicago, secretary.

Members of the committee are: W. C. E. Becker, consulting engineer, St. Louis; Frank H. Beinhauer, J. L. Simmons Co., Inc., Decatur, Ill.; Delmar L. Bloem, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.; Frank B. Brown, Wire Reinforcement Institute, Inc., Washington, D. C.; T. F. Collier, Westcott & Mapes, New Haven, Conn.; Joseph Di Stasio, consulting engineer, New York; Malcolm S. Douglas, Board of Building Standards and Building Appeals, Cleveland; A. Epstein, A. Epstein & Sons, Inc., Chicago; Phil M. Ferguson, University of Texas, Austin; E. I. Fiesenheiser, Illinois Institute of Technology, Chicago; Thor Germundsson, Portland Cement Association, Chicago; Eivind Hognestad, Portland Cement Association, Chicago.

Harry F. Irwin, Warner Co., Philadelphia; Robert A. Jameson, Thomas, Jameson & Merrill, Dallas; Robert C. Johnson, Siesel Construction Co., Milwaukee; Oliver G. Julian, Jackson and Moreland, Boston; Frank Kerekes, Michigan College of Mining and Technology, Houghton; George E. Large, Ohio State University, Columbus; Nolan D. Mitchell, National Bureau of Standards, Washington, D. C.; I. E. Morris, Morris, Boehmig and Tindel, Atlanta; Nathan M. Newmark, University of Illinois, Urbana; Douglas E. Parsons, National Bureau of Standards, Washington, D. C.; Orley O. Phillips, Phillips, Carter, Osborn, Inc., Denver; Theodore O. Reyhner, Chico State College, Chico, Calif.; Paul Rogers, consulting

MAKE RESERVATIONS EARLY

Reservations for the forthcoming ACI regional meeting in Seattle may be made directly to the Benjamin Franklin Hotel

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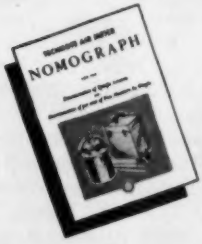

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structural engineer, Chicago; Robert Sailer, U. S. Bureau of Reclamation, Denver; Chester P. Siess, University of Illinois, Urbana; Howard Simpson, Massachusetts Institute of Technology, Cambridge; L. C. Urquhart, Porter, Urquhart, McCreary, and O'Brien, Newark, N. J.; A. C. Weber, Laclede Steel Co., St. Louis; C. H. Westcott, Westcott Engineering Co., Chicago; Walter H. Wheeler, consulting engineer, Minneapolis; C. A. Willson, American Iron & Steel Institute, New York; George Winter, Cornell University, Ithaca; H. B. Zackrison, Sr., Office of the Chief of Engineers, Department of the Army, Washington, D. C.

Explanations covering other sections of the code are expected to be prepared by the committee and will be published in forthcoming issues of the JOURNAL.

Bryant Mather

Originally presented at the ACI 53rd annual convention in Dallas, Feb. 27, 1957, Bryant Mather's paper "Laboratory Tests of Portland Blast-Furnace Slag Cements" now appears in print on p. 205.

Mr. Mather, a supervisory civil engineer with the Waterways Experiment Station, Corps of Engineers, U. S. Army, Jackson, Miss., is chief of the Special Investigations Branch, Concrete Division.

Receiving his degree in geology from Johns Hopkins University in 1936, he continued with graduate studies there and at American University, Washington, D. C., until 1941. He then joined the Corps of Engineers in concrete research.

He has just completed three terms as chairman of the ACI Technical Activities Committee and presently is a member of that committee as well as Committee 612, Recommended Practice for Curing Concrete, and Committee 212, Admixtures.

A member of ACI since 1944, Mr. Mather is also a member of ASTM, American Geophysical Union, Highway Research Board committees, and other professional societies; he is also secretary of ASTM Committee C-9.

F. T. Mavis and M. J. Greaves

"Destructive Impulse Loading of Reinforced Concrete Beams," on p. 255, has been co-authored by F. T. Mavis and M. J. Greaves.

F. T. Mavis is now dean of engineering, University of Maryland. Until recently he was head of the department of civil engineering at Carnegie Institute of Technology. Prior to 1944 he served as department head at the State University of Iowa and Pennsylvania State University. He was educated at the University of Illinois and at the Technische Hochschule in Karlsruhe where he was one of the first Freeman Traveling Fellows. Dean Mavis has written extensively in hydraulic engineering, structural analysis, and applied mechanics.

M. J. Greaves is presently engaged as development engineer for Arthur G. McKee & Co., Cleveland. He formerly was professor of civil engineering at Utah State Agricultural College. As a major in the Panama Canal Department during World War II he supervised military construction projects. His experience has included many unusual problems in reinforced concrete design and stress analysis. Graduating from Cornell University in 1940, he received his PhD from Carnegie Institute of Technology in 1956. He is a member of ASCE, ASME, and other professional societies.

LOOKING AHEAD

October 9-11, 1957—National Slag Association, 40th Annual Meeting, Plaza Hotel, Washington

October 14-18, 1957—American Society of Civil Engineers, National Convention, Statler Hotel, New York, N. Y.

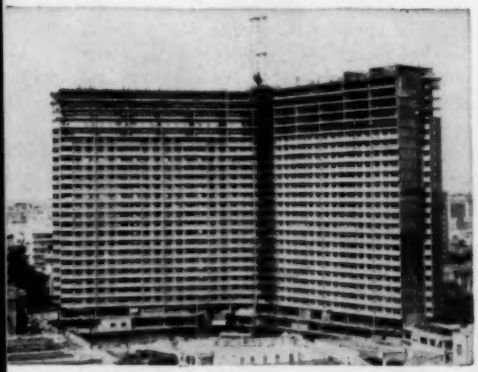
October 18-19, 1957—National Society of Professional Engineers, Fall Meeting, Grand Pacific Hotel and Provident Insurance Building, Bismarck, N. D.

November 5-6, 1957—ACI Regional Meeting, Benjamin Franklin Hotel, Seattle, Wash.

November 13-15, 1957—American Standards Association, Annual Meeting and National Conference on Standards, St. Francis Hotel, San Francisco, Calif.

General Hospital Neurological Building, Philadelphia; Archt. — Harbeson, Hough, Livingston & Larson; Contr. — McCloskey & Co., Inc.; Pozzolith Ready-Mixed Concrete — The Warner Co. — All of Philadelphia.

Prudential Insurance Bldg., Jacksonville, Fla.; Archt. — Kemp, Bunch & Jackson; Engr. — Reynolds, Smith & Hills; Contr. — Daniel Construction Co.; Pozzolith Ready-Mixed Concrete — Capitol Concrete Co. — all of Jacksonville.



"17 y M" — Reinforced concrete apartment building, Havana, Cuba; Archt. — Ernesto Gomez Sampera and Martin Dominguez; Struc. Engr. — Saenz-Cancio-Martin; In Charge of Construction: Bartolome Bestard and Manuel Padron.

Capitol Records Office Building, Hollywood, Calif.; Archt. Engr. — Welton Becket, F.A.I.A., and Assocs., L. A.; Struc. Engr. — Murray Erick Assocs., L. A.; Gen. Contr. — C. L. Peck, L. A.; Pozzolith Ready-Mixed Concrete — Transit-Mixed Concrete Co., Pasadena.

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Honor Roll

February 1—July 31, 1957

Blas Lamberti is leading the Honor Roll this month by proposing 14 new members giving him a total of 15 credits. G. B. Southworth is second with 14 credits and Arthur N. L. Chiu is third with 9.

With the increasing use of concrete it is important to keep up with the latest developments. You can do this by becoming a member of ACI and receiving the latest technical information available on the subject of concrete.

Blas Lamberti	15
G. B. Southworth	14
Arthur N. L. Chiu	9
Luis Perez Cid	6
Martin J. Gutzwiller	6
H. C. Pfannkuche	6
Phil M. Ferguson	5
Walter H. Price	5
Nicandro Barboza	4
W. S. Cottingham	4
Jaime de las Casas	4
Louis A. Gottheil	4
Henry L. Kennedy	4
L. M. Legatski	4
James A. McCarthy	4
Martin Schulz	4
S. E. Acker	3
James C. Amrheim	3
Luther E. Bell	3
Geo. F. Bishop	3
R. F. Blanks	3
C. A. Engman, Jr.	3
Ambrosio R. Flores	3
Myle J. Holley, Jr.	3
J. E. Jellick	3
Franklin B. Johnson	3
Robert H. Lochow	3
I. Martin	3
George H. Nelson	3
Howard Simpson	3
H. T. Williams	3
John T. Young	3
Miles N. Clair	2½
W. H. Armstrong	2
Jose L. Capacete	2
James Chinn	2
Dale Cobb	2
William V. Coyle	2
Roger D. de Cossio	2
C. F. DeVilbiss	2
E. I. Fiesenheiser	2
Kenneth H. Gedney	2
Roger E. Harlepp	2
N. L. Hinkson	2
Samuel Hobbs	2
M. W. Huggins	2
Leo Liberthson	2
Harry McLean	2
Patrick McNally	2
Frederick T. Mavis	2
Izuo Miyashita	2
John M. Muir	2
Richard A. Parmelee	2
Abdur Rahman S. Rasul	2
Theodore O. Reyhner	2
Sabri Sami	2
Wallace Sanders, Jr.	2
Harry Saxe	2
Harold Schweitzer	2
William C. Schwenger	2
Joaquin Spinel L.	2
J. P. Thompson	2
E. W. Thorson	2
Ellis S. Vieser	2
D. K. Woodin	2

New Members

The Board of Direction approved 74 Individual applications, 2 Corporations, 20 Juniors, and 6 Students, making a total of 102 new members. Considering losses due to deaths, resignations, and non payment of dues the total membership on Aug. 1, 1957, was 8939.

Individual

ARROYO, LOUIS M., Santurce, Puerto Rico (Struct. Designer, Torro, Ferrer, Archs.)

BALDERRAMA M., JAIME, Cerro de Pasco, Peru (C. E., Compania Minera Atacocha)

BORODAWKIN, DAVID, Tucson, Arizona (Engr., Blanton & Cole)

BROOKER, REV., W., Edmonton, Alta., Canada (Asst. to Chf. Engr., Arch. Branch, Gov't. of Alberta)

CAICEDO A., BERNARDO, Popayan, Colombia (C. E., Prof. V. del C., Chf. Engr., Tradie Ltda.)

CALLINAN, BERNARD J., East Melbourne, Victoria, Australia (Sr. Partner, Gutteridge, Haskins & Davey, Cons. Engrs.)

CHIN, F. Q., San Francisco, Calif. (Partner, Chin & Hensolt, Struct. Engrs.)

COLLEY, B. E., Chicago, Ill. (Sr. Development Engr., PCA)

CONNELL, JOHN W., Melbourne, Victoria, Australia (Cons. Engr., Cyril Hudspeth Assocs.)

CRADDOCK, JACK E., Hollywood, Calif. (Sales Repres., Monolith Portland Cement)

DA SILVA C., ELIAS G., Trujillo, Dominican Republic (C. E., Const. Dom. Del Conte & Allasia)

DIBOLL, COLLINS C., New Orleans, La. (Partner, Collins C. Diboll, Arch. Firm)

EKELUND, DR. BROR S., Madison, Wis. (Dir., Metallurgy & Ceramics Div., Trionics Corp.)

ENSLEN, E. R., Montgomery, Ala. (District Struct. Engr., PCA)

EVERHEART, E. F., Dallas, Texas (Sales Engr., Master Builders Co.)

FISCHER, ALBERT, Garches (Seto), France (Pres., Entreprise Generale Industrielle)

GELHARDT, HERBERT G., Tokyo, Japan (Engr., Asiatic Div., Master Builders)

GRIFFIN, JAMES C., Des Moines, Iowa (Plant Supt., Marquette Cement Mfg. Co.)

HAZARD, STUART G., Topeka, Kansas (Partner, Servis, Van Doren & Hazard)

HEATH, CHARLES F., JR., Hato Rey, Puerto Rico (Research Dir., Owner, Weldeon)

HENNING, NORMAN E., St. Paul, Minn. (Chf. Engr., Twin City Testing & Engrg. Lab., Inc.)

HOFF, ROBERT, Tel-Aviv, Israel (Dir. & Partner, J. Lipovits & R. Hoff Ltd.)

HOPE, JOHN DESMOND, Auckland, New Zealand (Refractory engr.)

ILLERICH, O. T., San Jose, Calif.

JONES, E. M., SR., Belvidere, Ill. (Matls. Engr., Joseph K. Knoerle & Assocs.)
KIMBERLING, LUTHER A., Harrisburg, Pa. (Managing Physical Testing Dept., Andrew S. McCreath Labs.)
KOENITZER, LESTER, Davis, Calif. (Matls. Engr., U. S. Corps of Engrs.)
KUS, JOHN, Detroit, Mich. (Struct. Engr., Hazen & Sawyer)
LEACH, WILLIAM O., Coleman, Texas (Cons. Engr., The Leach Engrs.)
LUSTIG, THEODOR HENRY, Mt. Lawley (Perth) Western, Australia (C. E., City Engineer's Dept., Perth City Council)
LUTHER, CHARLES C., Zulia, Venezuela (Head of Quality Control Unit, Creole Petroleum Corp.)
MALTEPE, HAKKI, Istanbul, Turkey (Stud. at Univ. of Texas)
MANZUR, JORGE, Cordoba, Argentina (C. E., Direccion Provincial de Hidraulica-Cordoba)
MAXWELL, WILLIAM R., Oklahoma City, Okla. (Salesman & Service Repres., The Master Builders Co., Div. of American-Marietta Co.)
MCKAGUE, J. V., Edmonton, Alta., Canada (Mgr., W. F., Macalister Ltd.)
MCNEIL, THOMAS J., New York, N. Y. (Eastern Mgr. & Public Relations Dir., Soiltest, Inc.)
MITCHELL, DAVID S., Baie Comeau, Que., Canada (Proj. Mgr., Anglin-Norcross Que.)
MORSY, NASHAAT, Cairo, Egypt (Cons., Arch. to the Courts & The Succession's Duty Dept.)
MOUHOT, ERNEST, JR., Lake Charles, La. (Struct. Designer, Dunn & Quinn, Archs. & Engrs.)
MULLANEY, JOSEPH W., JR., New York, N. Y. (Estimate & Spec. Engr., DeLeuw, Cather & Brill)
NALLAKRISHNAN, GOVINDAPPA, Urbana, Ill. (Design Engr., Clark, Daily & Dietz, Cons. Engrs.)
NIEBUR, ROBERT GERALD, Phoenix, Ariz. (Struct. Design & Drafting, Edward L. Varney & Assocs.)
OISHI, SATOSHI, Bayside, N. Y. (Struct. Engr., & Proj. Engr., Edwards, Kelcey & Beck)
ORETA, MAURO C., Manila, Philippines (Struct. Engr. & Constr. Engr., A. M. Oreta & Co., Inc.)
PAYET G., GUILLERMO, Lima, Peru (Arch., C. E., Contr.)
PEACOCK, ROBERT E., Maitland, Fla. (Research & Control Engr., Holloway Concrete Products Co.)
PIERCE, O. F., Rillito, Ariz. (Plant Chemist, Arizona Portland Cement Co.)
POKORNY, JAROSLAV V., Houston, Tex. (Sr. Draftsman, Texas Highway Dept.)
PATTEN, NORMAN B., Los Angeles, Calif. (Chf. Struct. Engr., Albert C. Martin & Assocs.)
PROUD, STANLEY L., New York, N. Y. (Engr., Mtls., Arch., Engr., Keflavik Airbase, Iceland)
ROBSON, ROBERT A., Welkom, OFS, South Africa (Gen. Mgr., Rocla Pipes (Africa) (Pty) Ltd.)
SCHNARR, RICHARD A., St. Paul, Minn. (Asst. City Arch., City of St. Paul, Minn.)
SIMPSON, DAVID R., Akron, Ohio (Cons. Engr. specializing in Struct. Field)
SLOCUM, WILLIAM R., Minneapolis, Minn. (Engr., Willis A. Jacus, Cons. Engr.)
SMITH, RAYMOND F. J., Welland, Ont., Canada (C. E., Design & Field Insp., Arthur B. Scott Assoc.)
SOLLID, ERIK, Washington, D. C. (Supv. Struct. Engr., American Embassy TCM, New Delhi, India)
SOVAK, R. J., Beaconsfield West, Quebec (Exec. Engr., Anglin-Norcross Corp. Ltd.)
STEINBERG, HAROLD B., Kansas City, Mo. (Struct. Designer, Cooper, Robison, Carlson & O'Brien, Archs., Engrs.)
STELSON, THOMAS E., Pittsburgh, Pa. (Asst. Prof. of C. E., Carnegie Institute of Technology)
TAYLOR, PAUL G., Philadelphia, Pa. (Partner, McCormick-Taylor Assocs., Cons. Engrs.)
TORRES, MANUEL E. GONZALEZ, Caparra Terrace, Puerto Rico (Struct. Designer, Schimmelpfenning, Ruiz & Gonzales)
TUCKER, H. LEO, Abilene, Tex. (Pres., Tucker & Lindberg)
TUMA, JAN J., Stillwater, Okla. (Prof. of C. E., Stillwater University)
ULIGNY, S. M., Allison Park, Pa. (Struct. Engr., Highway Div., Swindell-Dressler Corp.)
VAN DEN BEMT, J., Moe, Victoria, Australia (Engrg. Asst., Civil Constr. Branch, Vallourn, Supv. Gen. Constr., State Electricity Comm. of Victoria)
VENEGAS MARTINEZ, RAMON, Caracas, Venezuela (C. E., Lusant S. A. Construcciones)
VILLAZON, JULIO, Caracas, Venezuela (Engr., DOF, C. A., DE Construcciones)
VINAYAKA, M. R., Bombay State, India (Research Officer, Koyna Project, Koyna P. O., Dist: N. Satara)
VOZORIS, S. S., Toronto, Ont., Canada (Head of Firm, S. S. Vozoris Cons. Struct. Engrs.)
WALLACE, LAWRENCE L., Detroit, Mich. (Chf. Struct. Engr., O. Germany & Assocs.)
WELTON, HERBERT A., Sioux City, Iowa (Director of Constr. Div., W. A. Klinger, Inc.)
WILKES, LUCIAN W., Pittsburgh, Pa. (Research Engr., Dravo Corp.)
YOUNG, EDUARDO, Lima, Peru (Struct. Engr., Prestressed Concrete)
ZALDUA, DR. ROQUE, Caracas, Venezuela

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UNIVERSIDAD DE CARTAGENA, Facultad de Ingenieria Civil, Cartagena, Colombia (Dr. Jose Antonio Coud)

Junior

AIKMAN, ALASTAIR JOHN MATHERS, Toronto, Ont. Canada (Jr. Engr., Hydro Electric Power Comm. of Ont.)
ANIHAL, FRED R., Virginia Beach, Va. (Ltjg. CEC, USNR, Asst. Pub. Wks. Officer)
BATIEVSKY, GEORGE, Lima, Peru (C. E., Constrs.)
BURCHETT, KNOX R., Lexington, Ky. (Research Engr., Ky. Dept. of Hwys.)
GARCIA DIAZ, FELIX R., Cda. Trujillo, Dominican Republic (C. E., Const. Dom. Del Conte & Allasia)
HAMILTON, JAMES W., JR., Chicago, Ill. (Struct. Engr., Skidmore, Owings & Merrill)
HARTDEGEN, WILLIAM J., Brooklyn, N. Y. (Struct. Engr., Roberts & Schaefer Co.)
HICKERSON, ARTHUR DEAN, Cleveland, Ohio (C. E., McDowell Co., Inc.)
HOFFMAN, HOWARD ROBERT, Bloomfield, Conn. (Struct. Engr., A. J. Macchi)
KING, HARRY LEON, JR., Atlanta, Ga. (Engr. in Trng, W. H. Armstrong)
LITLE, WILLIAM A., Cambridge, Mass. (Grad. Stud. MIT)
LORD, JEAN, Notre-Dame De grace, Montreal, Canada (Struct. Engr., Cartier, Cote & Piette, Cons. Engrs.)
MAHMOOD, KHALID, Lahore, West Pakistan (Asst. Exec. Engr., West Pakistan Gov't in Pub. Wks. Dept., Irrigation Branch)
QUEIROZ, LINCOLN ARAUJO, Belo Horizonte-Minas Gerais, Brasil (C. E., Centrais Eletricas Minas Gerais S/A)
RENASCO, MANUEL, JR., Montpelier, Vt. (Cons. Struct. Engr.)
ROUSE, MARTIN S., Houston, Tex. (Struct. Design & Drafting, Brown & Root, Inc.)
SANDERS, RONALD V., Jackson, Mich. (Engr. in Trng., Commonwealth Assocs., Inc.)
SEIDENSTICKER, JOHN F., Evanston, Ill. (Struct. Designer, Childs & Smith, Archs. & Engrs.)
TIJERINO M., ELPIDIA, Managua, DN, Nicaragua (C. E., Servicio Tecnico Agricola de Nicaragua)
VUKOVITZ, JACK, Chicago, Ill. (Jr. Struct. Designer, Skidmore-Owings & Merrill)

Student

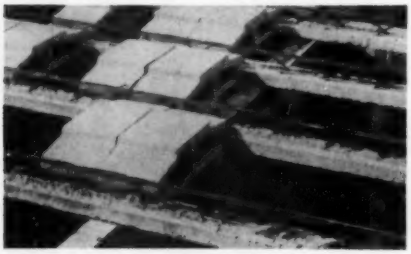
CERNICA, JOHN N., Youngstown, Ohio (Carnegie Inst. of Tech.)
DUTSCHMANN, WILBERT C., Port Arthur, Tex. (Univ. of Texas)
GABLE, ALDEN B., Angola, Ind. (Tri-State College)
HAMANAKA, HISAO, Angola, Ind. (Tri-State College)
TRAVERSE, JOHN DEREK, Champaign, Ill. (Univ. of Ill.)
VOEGELI, DONALD HENRY, Chicago, Ill. (Tri-State College)

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

New concrete floor system

A type of concrete floor and roof deck developed in Germany, said to combine structural and erection advantages of cast-in-place and precast methods, is now available in the United States. Called the Omnia



floor, the block-and-beam deck is adaptable to spans of 30 ft or more. The floor consists of three components: the Omnia "plank" (reinforced steel lattice girder with reinforced concrete base), filler block (lightweight concrete), and concrete topping. The planks and block provide both support and forming for the concrete topping.—Omnia Construction Corp., 30 South Broadway, Yonkers, N. Y.

Flexible metal hose

Recently published catalog (ID-100A) covering wide range of flexible metal hose manufactured by Universal Metal Hose Co. may be procured from the manufacturer. Completely describes and illustrates various types, styles, and sizes.—Universal Metal Hose Co., 2133 South Kedzie Ave., Chicago 23, Ill.

Test cylinder carrier

Designed to hold two test cylinders, the Bristol concrete cylinder carrier affords greater ease in handling. Test cylinders can be carried immediately after being filled by slipping ring portion of carrier down over top of cylinder, tipping slightly so that bottom right angle hooked portion of carrier will go under cylinder.—Mark Bristol, 18137 Lamson Road, Castro Valley, Calif.

Perimeter insulation

The characteristics and specifications of Perimsul, a perimeter insulation felted from spun mineral wool, are described in a 4-p. bulletin published by Baldwin-Hill Co.

In addition to detailing the properties of the material, the brochure also describes the need for perimeter insulation under concrete slabs and in foundation crawl spaces, and explains and illustrates five methods of applying perimeter insulation.—Baldwin-Hill Co., 221 Breunig Ave., Trenton, N. J.

Pneumatic concrete applicator

The Aircreter, which pneumatically applies concrete and refractory materials, is now offered by Engineered Equipment, Inc. Handling sand, aggregates, and refractory materials, dry or up to 5 percent moisture content, this gun places 1 to 6 cu yd per hr according to the manufacturer. Pressure is controlled by air regulator. It is available in portable models for patching and repairing jobs and large models for production work.—Engineered Equipment, Inc., Waterloo, Iowa

New tank truck system

A new tank truck system for metered delivery of a ready-to-use, catalyzing type, liquid water-reducing agent for concrete called WRDA, has been introduced by Dewey and Almy Chemical Co. According to the manufacturer, WRDA reduces the amount of water needed in concrete by an average of 15 percent by plasticizing the concrete mix. With the metered tank truck delivery system, ready-mixed concrete plants are supplied with routine deliveries when the supply runs low.—Dewey and Almy Chemical Co., 62 Whittemore Ave., Cambridge 40, Mass.

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Galvanized wire reinforcing

Wal-Lok horizontal mortar joint reinforcing is now available fabricated from galvanized wire. Material used is 100,000 psi cold-drawn steel which has been "electrolytically galvanized."—Adrian Peerless, Inc., 1428 E. Michigan St., Adrian, Mich.

Drill starting guides

Drill starting guides are now offered free of charge to users of Tilden Rotary Konkrete Kore Drills by Tilden Tool Manufacturing Co. The drill guide has guide holes for drills ¼ through 1¼ in. diameter, providing a means of precision locating and starting of hole with ease. Cost of starting drills for core drills is eliminated by use of this guide according to manufacturer.—Tilden Tool Manufacturing Co., 209 Los Molinos, San Clemente, Calif.

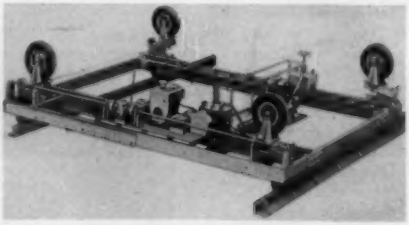
Determining moisture of block

An instrument for determining the moisture condition of hardened concrete block by the relative humidity method has been introduced by Central Scientific Co. The manufacturer states that the CENCO moisture-humidity apparatus, in a period of 30 min or less, will indicate whether concrete block are dry enough or require further drying to minimize drying shrinkage during exposure to average relative humidity.—Central Scientific Co., 1700 Irving Park Road, Chicago, Ill.

Pavement float finisher

A new line of Rex longitudinal float finishing machines in models from 10- to 32-ft working widths and offering numerous design and operating advancements has been announced by Chain Belt Co.

Speed of operation is stressed by the manufacturer in these new models. Powered-frame widening permits easy nonstop changes in widths to speed work and



reduce costs in interchange and tapered-lane work. Hydraulic pedestals at each corner lift the finisher off forms without blocking or need of special tools. The large float measure 12 ft in length and has up to 12 in. lag for fast working under all conditions. All controls are grouped for efficient operation and maximum visibility. Removable side rails permit collapsing machine for trailing on transportation wheels or truck loading. —Chain Belt Co., Milwaukee 1, Wis.

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